GEOTECHNICAL INVESTIGATION

NORTHWEST WASTEWATER TREATMENT PLANT IMPROVEMENTS
WBS NO. R-000265-0095-4
5423 MANGUM ROAD
HOUSTON, TEXAS

Reported To:
Parsons, Inc.
Houston, Texas

by

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REPORT NO. G178-13

March 2014

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March 26, 2014

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Reference: **Geotechnical Investigation**
Northwest Wastewater Treatment Plant Improvements
WBS No. R-000265-0095-4
5423 Mangum Road
Houston, Texas
AEC Report No. G178-13

Dear Mr. Thornber,

Aviles Engineering Corporation (AEC) is pleased to present this report of our geotechnical investigation for the above referenced project. This investigation was authorized on December 3, 2013 by Mr. Rick W. Miller, Contracts & Procurement Manager of Parsons, Inc., based upon AEC Proposal No. G2013-11-09R, dated November 21, 2013. This final report will supersede the previously submitted reports and transmittals for the referenced project.

AEC appreciates the opportunity to be of service to you. Please call us if you have any questions or comments concerning this report or when we can be of further assistance.

Respectfully submitted,
Aviles Engineering Corporation
(TBPE Firm Registration No. F-42)

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EXECUTIVE SUMMARY

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed improvements at the City of Houston's (COH) Northwest Wastewater Treatment Plant (WWTP), located at 5423 Mangum Road, in Houston, Texas (Houston/Harris Key Map: 451C and D). Based on the information provided, AEC understands that proposed improvements consist of (i) replacement of eight existing valves located 25 feet below grade in the clarifier area; (ii) adding two vertical chemical storage tanks just south of the chemical building; (iii) a new horizontal hydropneumatic pressure tank located to the north of the chlorine contact basin; (iv) adding an electrical building and a fan building to the northwest portion of the WWTP; and (v) a new scum separation pad located to the southwest of Clarifier #2.

1. Subsurface Soil Conditions: The subsurface soils encountered in our borings at the site generally consist of 2 feet to 15 feet of clayey sand (SC), sandy silt (ML), sandy lean clay (CL) fills, underlain by very loose to dense poorly graded sand w/silt (SP-SM), poorly graded sand (SP), silty sand (SM), silty clayey sand (SC-SM), clayey sand (SC), and silt (ML) to the boring termination depths. Very stiff to hard clays were encountered at depths of approximately 6 to 12 feet in Borings B-4 through B-6.

2. Subsurface Soil Properties: The subsurface clayey soils have high plasticity, with liquid limits (LL) ranging from 35 to 45, and plasticity indices (PI) ranging from 21 to 31. High plasticity clays can undergo significant volume changes due to seasonal changes in moisture contents. The cohesive soils encountered are classified as "CL" type soils and granular soils were classified as "SC", "SM", "SP", "SP-SM", and "ML" in accordance with ASTM D 2487.

3. Groundwater Conditions: Groundwater was encountered at the depths of 22 to 23 feet below existing grade during drilling in Borings B-1, B-2, B-5, and B-6. Ground water was not encountered in the remaining borings during drilling. Approximately 24 hours after completion of drilling, groundwater levels at Borings B-1 and B-2 were taken at 20.3 feet and 18.8 feet, respectively.

4. Hazardous Materials: No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

5. Design soil parameters and recommendations for internally braced sheet pile walls for the valve pits are presented in Sections 5.1 of this report.

6. Recommendations for feasible foundation options for the buildings, tank pads, and scum preparation pad are presented in Sections 5.2.1 and 5.2.2 of this report.

This Executive Summary is intended as a summary of the investigation and should not be used without the full text of this report.

GEOTECHNICAL INVESTIGATION

NORTHWEST WASTEWATER TREATMENT PLANT IMPROVEMENTS
WBS NO. R-000265-0095-4
5423 MANGUM ROAD
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 General

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed improvements at the City of Houston's (COH) Northwest Wastewater Treatment Plant (WWTP), located at 5423 Mangum Road, in Houston, Texas (Houston/Harris Key Map: 451C and D). A vicinity map is presented on Plate A-1 in Appendix A. Based on the information provided, AEC understands that the proposed improvements consist of (i) replacement of eight existing valves located 25 feet below grade in the clarifier area; (ii) adding two vertical tanks just south of the chemical building; (iii) a new horizontal hydropneumatic pressure tank located to the north of the chlorine contact basin; (iv) adding an electrical building and a fan building at the northwest portion of the WWTP; and (v) a new scum separation pad located to the southwest of Clarifier #2.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil and groundwater conditions and develop geotechnical engineering recommendations for the improvements. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling 6 geotechnical borings, ranging from 16 to 40 feet below existing grade;
2. Soil laboratory testing on selected soil samples;
3. Engineering analyses and recommendations for shoring systems, bedding and backfill for valve pits excavation;
4. Engineering analyses and recommendations for feasible foundation types and depths, and allowable bearing capacities for the buildings, tank pads, and scum separation pad; potential vertical rise, and recommendations for floor slab and subgrade preparation for the buildings;
5. Geotechnical recommendations and dewatering guidelines for the facility construction.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions at the site were investigated by drilling six borings to depths ranging from 16 to 40 feet below existing grade. The total drilling footage was 176 feet. Boring B-3 was terminated at a depth of 16 feet during drilling after an unknown obstruction was encountered. After completion of drilling, the borings were surveyed by others. Boring survey data is included on the representative boring logs. The boring locations are shown on the Boring Location Plan presented on Plate A-2, in Appendix A. A summary of borings performed for this project are presented on Table 1.

Table 1. Summary of Borings

Boring No.	Boring Depth (ft)	Boring Purpose	Mat Dimension[b] (ft)	Structural Loading[c] (psf)
B-1	40	Valve Replacement	-	-
B-2	40	Valve Replacement	-	-
B-3[a]	16	Fan Building	B=15;L=18	278
B-4	20	Electrical Building	B=15;L=20	250
B-5	30	Horizontal Tank	B=18;L=28	83
B-6	30	Two Vertical Tanks	B=20;L=28	34

Notes: (a) Boring terminated when unknown obstructed was encountered. Mr. Robert Thornber, P.E., of Parsons, Inc., indicated that B-3 was drilled within the Harris County Flood Control District through concrete pavement next to an existing building that will be demolished and replaced. There will be no increase in impervious cover.
(b) B=pad width; L=pad length.
(c) Structural loading was provided by Mr. Robert Thornber, P.E., of Parsons, Inc.

The borings were drilled using a truck-mounted drill rig; existing concrete pavement at Boring B-3 was cut with a core barrel prior to drilling. Borings were performed initially by dry auger method, then using wet rotary method once the borings caved in or saturated granular soils were encountered. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in accordance with ASTM D 1587. Granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D 1586. Standard Penetration Test resistance (N) values were recorded for the granular soils as “Blows per Foot” and are shown on the boring logs. Strength of the cohesive soils was estimated in the field using a hand penetrometer. The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. After completion of drilling, Borings B-1 and B-2 were left open for a period of approximately 24 hours so that an additional groundwater reading could be taken. The borings were grouted with cement-bentonite and the existing pavement at Boring B-3 was patched with non-shrink grout. Details of the soils encountered in the borings are presented on Plates A-3 through A-8, in Appendix A.

3.0 LABORATORY TESTING PROGRAM

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under the supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, percent passing a No. 200 sieve, sieve analysis, and dry unit weight tests were performed on typical samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were determined by means of unconfined-compression (UC) and undrained-unconsolidated (UU) triaxial tests performed on undisturbed samples. The test results are presented on the boring logs. Details of the soils encountered in the borings are presented on Plates A-3 through A-8, in Appendix A. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates A-9 through A-12, in Appendix A. Sieve analysis test results are presented on Plate A-13, in Appendix A. A summary of laboratory test results is presented on Plates A-14 and A-15, in Appendix A.

4.0 SITE CONDITIONS

4.1 Subsurface Conditions

Soil strata encountered in our borings are summarized below.

Boring	Depth	Description of Stratum
B-1	0’ - 15’	Fill: Clayey Sand (SC) with roots, shell fragments, calcareous and ferrous nodules, and brick pieces
	15’ - 22’	Medium dense, Poorly Graded Sand w/Silt (SP-SM)
	22’ - 25’	Medium dense, Poorly Graded Sand (SP)
	25’ - 30’	Medium dense, Silty Sand (SM)

Boring	Depth	Description of Stratum
B-2	0’ - 10’	Fill: Clayey Sand (SC) with roots, shell and siltstone fragments, ferrous and calcareous nodules
	10’ - 16’	Fill: loose, Clayey Sand (SC)
	16’ - 40’	Very loose to medium dense, Poorly Graded Sand w/Silt (SP-SM)
B-3	0’ – 0.65’	Concrete Pavement
	0.65’ - 4’	Fill: Clayey Sand (SC) with shell and siltstone fragments, calcareous nodules
	4’ - 10’	Fill: Silty Clayey Sand (SC-SM)
	10’ - 16’	Very loose to loose, Silty Clayey Sand (SC-SM)
B-4	0’ - 2’	Fill: Sandy Silt (ML)
	2’ - 6’	Fill: stiff to very stiff, Sandy Lean Clay (CL), with crushed base and siltstone fragments
	6’ - 10’	Very stiff to hard, Sandy Lean Clay (CL)
	10’ - 16’	Loose to medium dense, Clayey Sand (SC)
	16’ - 20’	Loose to medium dense, Poorly Graded Sand w/Silt (SP-SM)
B-5	0’ - 6’	Fill: Stiff to hard, Sandy Lean Clay (CL) with roots, calcareous and ferrous nodules, siltstone fragments
	6’ - 12’	Very stiff, Sandy Lean Clay (CL), with calcareous and ferrous nodules, siltstone fragments
	12’ - 30’	Medium dense to dense, Silty Sand (SM)
B-6	0’ - 2’	Fill: hard, Sandy Lean Clay (CL) with roots, shell fragments, ferrous nodules
	2’ - 6’	Very dense, Clayey Sand (SC), with roots
	6’ - 10’	Hard, Sandy Lean Clay (CL), with ferrous nodules
	10’ - 14’	Dense, Silty Sand (SM)
	14’ - 16’	Dense, Silt (ML)
	16’ - 30’	Dense to very dense, Poorly Graded Sand w/Silt (SP-SM)

Subsurface Soil Properties: The subsurface clayey soils have high plasticity, with liquid limits (LL) ranging from 35 to 45, and plasticity indices (PI) ranging from 21 to 31. High plasticity clays can undergo significant volume changes due to seasonal changes in moisture contents. The cohesive soils encountered are classified as “CL” type soils and granular soils were classified as “SC”, “SM”, “SP”, “SP-SM”, and “ML” in accordance with ASTM D 2487. “CH” soils undergo significant volume changes due to seasonal changes in soil moisture contents. “CL” type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, “CL” soils with LL approaching 50 and PI greater than 20 essentially behave as “CH” soils and could undergo significant volume changes.

Groundwater Conditions: Groundwater was encountered at the depths of 22 to 23 feet below existing grade during drilling in Borings B-1, B-2, B-5, and B-6. Ground water was not encountered in the remaining borings during drilling. Approximately 24 hours after completion of drilling, groundwater levels at Borings B-1 and B-2 were taken at approximately 20 feet and 19 feet, respectively. Groundwater at the site could be pressurized. Groundwater levels encountered are summarized in Table 2.

Table 2. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth Encountered during Drilling (ft)	Groundwater Depth after Completion of Drilling (ft)	Groundwater Depth after 24 Hours (ft)
B-1	12/18/2013	40	22	20 (cave in)	20
B-2	12/18/2013	40	23	22 (cave in)	19
B-3	12/18/2013	16	Dry	Dry	--
B-4	12/18/2013	20	Dry	Dry	--
B-5	12/19/2013	30	23	23 (cave in)	--
B-6	12/19/2013	30	23	23 (cave in)	--

The information in this report summarizes groundwater conditions observed on the dates the borings were drilled. It should be noted that our groundwater observations are short-term; groundwater depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Hazardous Materials

No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

4.3 Subsurface Variations

It should be emphasized that: (i) at any given time, groundwater depths can vary from location to location, and (ii) at any given location, groundwater depths can change with time. Groundwater depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary away from and between the boring locations.

Clay soils in the Houston area typically have secondary features such as slickensides and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples which were generally continuously obtained at intervals of 2 feet from the ground surface to a depth of 20 feet, then at 5 foot intervals thereafter to the boring termination depths. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while some of AEC's logs show the soil secondary features, it should not be assumed that the features are absent where not indicated on the logs.

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Based on the information provided, AEC understands that the proposed improvements consist of (i) replacement of eight existing valves located 25 feet below grade in the clarifier area; (ii) adding two vertical tanks just south of the chemical building; (iii) a new horizontal hydropneumatic pressure tank located to the north of the chlorine contact basin; (iv) adding an electrical building and a fan building at the northwest portion of the WWTP; and (v) a new scum separation pad located to the southwest of Clarifier #2.

Mr. Robert Thornber, P.E., of Parsons, Inc. provided structural loading for the proposed fan building, electrical building, horizontal hydro tank, and two vertical tanks. In addition, Mr. Thornber indicated that the proposed scum pad has light equipment weights. This loading information has been summarized in Table 1 in Section 2.0 of this report.

5.1 Clarifier Valve Replacements

According to the information provided, eight existing valves located 25 feet below grade in the clarifier area will be replaced. Due to the limited space in the area, AEC recommends braced steel sheeting piles be used in each excavation pit.

5.1.1 Internally Braced Steel Sheet Piling

Based on the subsurface soil conditions in Borings B-1 and B-2 and our experience with similar projects, we recommend that internally braced interlocking steel sheet piles be designed based on the following conditions: (i) groundwater level at 22 feet behind the sheet piles; (ii) a worst case combination of live and dead load surcharges (including construction equipment, stockpile, and adjacent clarifier loads); and (iii) a Factor of Safety (FS) of 2 used for passive earth pressure resistance in front of the sheet piles.

Lateral Earth Pressures: The magnitudes of the lateral earth pressures will depend on the type and density of the retained soils, surcharge on the retained soils, adjacent clarifier loads on the retained soils, and hydrostatic pressure. Lateral pressure resulting from construction equipment, pavement and traffic, or other surcharge on the top of the sheet piles should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. We recommend that at least a 240 psf construction surcharge be considered for design of the walls. Depending on the proximity of the adjacent clarifier(s) to the excavation area, the clarifier(s) loads should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure.

Sheet pile walls can be designed based on active earth pressures. The active earth pressure at depth z can be determined by Equation (1).

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \quad \text{............Equation (1)}$$

where,

p_a	=	active earth pressure, psf.
K_a	=	coefficient of active earth pressure, see Plates B-1 and B-2, in Appendix B.
c	=	cohesion of clayey soils (can be conservatively neglected for design), see Plates B-1 and B-2, in Appendix B.
q_s	=	uniform surcharge pressure, psf.
γ, γ'	=	wet and buoyant unit weights of soil, pcf, see Plates B-1 and B-2, in Appendix B.
h_1	=	depth from ground surface to groundwater table, feet.
h_2	=	$z-h_1$, depth from groundwater table to point under consideration, feet.
z	=	depth below ground surface, feet.
γ_w	=	unit weight of water, 62.4 pcf.

Design soil parameters for the wall design are presented on Plates B-1 and B-2, in Appendix B. The design values are based on the results of field and laboratory test data on individual boring logs as well as our experience. It should be noted that because of the variable nature of soil stratigraphy, soil types and properties at locations away from a particular boring may vary substantially.

Internal Bracing: Internal bracing is used to transfer lateral earth (and water pressures) between opposing sheet piling walls through compressive struts. Typically the struts are either pipe or I- beam sections and are usually preloaded to provide a very stiff system (i.e. less movement). Installation of the bracing struts is done by excavating soil locally around the strut and only continuing the excavation once preloading is complete. The struts should connect to wale beams that distribute the strut load horizontally to the sheet piling walls.

Ground Movement and Monitoring: Since the excavation will be close to existing clarifiers and underground pipelines, the design engineer should evaluate their tolerance to movement (i.e. lateral and vertical movements). Retained soils may also settle due to extended dewatering, if implemented.

Preloading struts is an effective way to reduce lateral wall displacement. During excavation, potential bottom heave, lateral wall movement, and settlement of areas behind the walls should be inspected regularly and carefully monitored as necessary.

Bottom Stability: It is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by sheet pile cutoff or dewatering the area.

Based on the proposed excavation depth at 25 feet below grade and the groundwater conditions in Borings B-1 and B-2 (see Table 2 in Section 4.1 of this report), excavations may encounter ground water at approximately 22 feet below grade. If the excavation extends below groundwater and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 3 feet below the excavation.

Backfill Material: Backfill should be in general accordance with Section 02320 of the latest edition of the City of Houston Standard Construction Specifications (COHSCS). Backfill should be placed in loose lifts not exceeding 8 inches and compacted to 95 percent of its ASTM D-698 (Standard Proctor) maximum dry density at a moisture content ranging between optimum and 3 percent above optimum.

5.1.2 Sheet Pile Construction

The Contractor may need to dig a trench along the sheet pile walls to remove underground obstructions (such as concrete blocks), if any, prior to sheet pile installation.

Selection of driving equipment, construction and monitoring of the steel sheet piles should be performed by qualified personnel who are experienced in this operation. We recommend that the steel sheet piles be driven in pairs. It is important that the sheet pile with the ball end be driven first. If the sheet pile with the socket end is driven first, it may clog with soil and make it difficult to drive the adjacent pile. Regular inspection of sheet pile tops should be performed to assess damage resulting from driving through relatively hard soils. Construction of the sheet piles can be in accordance with the American Society of Civil Engineers (ASCE) publication "Design of Sheet Pile Walls" (1996), Chapter 8, or Item 408 of the 2012 Harris County Public Infrastructure Department (HCPID) "Standard Engineering Design Specifications for Construction and Maintenance of Roads and Bridges". Excavation with sheet piling can be in accordance with OSHA Subpart P, 1926.652(c), (d), (e), and (g).

5.2 Fan Bldg, Electrical Bldg, Scum Pad, Horizontal Tank, and Two Vertical Tanks

Based on our experience with previous improvements at the plant, the subsurface soils across the site consist of thick non-uniform and uneven fills. This is consistent with our findings in this investigation that the thickness of the fill soils varies from 2 to 10 feet. Large differential settlements are usually expected from foundations supported on such soils. Compared to spread footings, mat foundations can be designed to achieve higher structural rigidity and tolerate larger deflections when large differential settlements take place. Therefore, AEC recommends that mat foundations be used to support the proposed structures.

Alternatively, straight-sided drilled shafts can be used to support the proposed structures. However, sandy and silty soils were encountered between the depths of 0 and 40 feet across the site. Construction of drilled shafts in sandy soils usually requires temporary casing and/or bentonite slurry to maintain control over stability of the hole, which may not be economical enough to justify the extra construction effort.

5.2.1 Option 1: Mat Foundation

Mat Foundation: A summary of net allowable bearing capacity for sustained loads and total loads, based on a minimum FS of 3 and 2, respectively, is shown in Table 3. For each case, whichever load condition that is critical should be used for design.

Table 3. Summary of Allowable Bearing Capacity

Building Name (Reference Soil Boring)	Mat Dimension (ft)	Minimum Footing Depth (ft)	Minimum Footing Elevation (ft)	Allowable Bearing Capacity for Sustained Loads (FS=3) (psf)	Allowable Bearing Capacity for Total Loads (FS=2) (psf)
Fan Building (B-3)	B=15;L=18	3	66.02	2,000	3,000
Electrical Building (B-4)	B=15;L=20	3	70.32	1,800	2,700
Scum Separation Pad (B-1 and B-4)	B=22;L=35	3	Unknown	1,800	2,700
Horizontal Tank Pad (B-5)	B=18;L=28	3	70.17	2,000	3,000
Two Vertical Tank Pad (B-6)	B=20;L=28	3	70.59	2,000	3,000

Note: B=pad width; L=pad length.

Mat Settlement: A detailed settlement analysis is beyond the scope of this investigation. Based on the soil conditions encountered, we estimate that a mat foundation, designed and constructed as recommended in this report, will experience total settlements on the order of 1 inch.

Modulus of Subgrade Reaction: The modulus of subgrade reaction (k) is frequently used in the structural analysis of mat foundations. Based on the soil conditions encountered and the size of the mat foundation, we recommend using k = 50 pounds per cubic inch (pci) for a mat foundation founded at 3 feet below finished grade.

Subgrade Preparation: Subgrade preparation should extend a minimum of 5 feet beyond the mat slab perimeter. A minimum of 6 inches of surface soils, existing vegetation, trees, roots, soft/weak soils, and other deleterious materials shall be removed and wasted. The excavation depth should be increased when inspection indicates the presence of soft/weak soils, organics, and deleterious materials to greater depths.

Afterwards, an additional 2.5 feet of existing fill soils (total depth of 3 feet, which includes the 6 inches of surface removal) should be removed. The exposed subgrade should be proof-rolled in accordance with Item 216 of the 2004 Texas Department of Transportation (TxDOT) Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with compacted select fill. After proof-rolling, compacted select fill should then be used to achieve the design grades. Select fill should be in accordance with Section 5.3 of this report.

Alternatively, if the mat pad is embedded less than 3 feet deep below ground surface, we recommend that the top 8 inches of the exposed subgrade (after surface soils removal of 6 inches or more) be stabilized with either hydrated lime or a combination of hydrated lime and fly ash, depending on the soil conditions. Exposed clay soils (with a PI greater than 10) should be stabilized with a minimum of 5 percent hydrated lime by dry soil weight; whereas sands, silts, and sandy lean clays (with a PI of 10 or less) should be stabilized with at least 3 percent hydrated lime and 7 percent fly ash by dry soil weight. Lime and lime/fly-ash stabilization shall be performed in accordance with Sections 02336 and 02337 of the latest edition of the COHSCS, respectively. The percentage of lime and lime/fly-ash required for stabilization are preliminary estimates for planning purposes only; laboratory testing should be performed to determine optimum contents for stabilization prior to construction. The stabilized soils should be compacted to 95 percent of their ASTM D 698 (Standard Proctor) dry density at a moisture content ranging from optimum to 3 percent above optimum.

5.2.2 Option 2: Straight-Sided Drilled Shaft

Since an underground obstruction was encountered at a depth of 16 feet in Boring B-3, AEC does not recommend using straight-sided drilled shafts to support the proposed Fan Building.

5.2.2.1 Straight-Sided Drilled Shaft

The other proposed structures can be supported on straight-sided drilled shafts. We performed drilled shaft analyses using O'Neill and Reese's method, "Drilled Shaft Design and Construction" (1999). In the analyses, we neglected skin friction from the existing ground surface to 5 feet below existing grade because of the potential for desiccation and shrinkage of near surface clays, and/or potential construction disturbance. We used a Factor of Safety (FS) of 2 and 3 for skin friction and end bearing, respectively.

The total allowable compressive axial bearing capacity of a straight-sided drilled shaft is the sum of the allowable skin friction (obtained by multiplying the shaft perimeter by the allowable unit cumulative skin friction beginning from 5 feet below existing grade to the design depth) and the allowable end bearing (obtained by multiplying the shaft cross-sectional area by the allowable unit end bearing at the design depth). The allowable accumulative unit skin friction capacity vs. depth curve, allowable unit end bearing vs. depth curve, and allowable compressive load vs. depth curve for 24-, 36-, and 48-inch diameter straight-sided drilled shafts for the Electrical Building and Scum Separation Pad, Horizontal Hydropneumatic Tank Pad, and Two Vertical Chemical Storage Tanks Pad, are presented on Plates C-1 through C-3, respectively, in Appendix C.

When a shaft tip is terminated in a strong layer underlain by a weak layer and the thickness between the shaft tip and the top of the weak layer is less than 2 shaft diameters (feet), we recommend the use of the design end bearing of the lower weak layer.

Drilled Shaft Spacing: To reduce the influence of adjacent drilled shafts and group effects, the minimum center-to-center spacing between adjacent shafts should be at least 3 times the diameter of the larger shafts; the minimum edge-to-edge spacing between adjacent shafts should not be less than 3 feet. For a drilled shaft group with a pier cap in contact with the ground, the individual capacity of the drilled shaft should be multiplied by an efficiency factor, η, where $\eta = 0.7$ for a center-to-center spacing of 3D (D is the diameter of the larger shaft) and $\eta = 1.0$ for a center-to-center spacing of 6D. The value of η may be linearly interpolated for intermediate spacing. The group capacity will be the smaller of: (i) the sum of the individual capacities of the drilled shaft multiplied by η, or (ii) the bearing capacity for the block (i.e., the group of the drilled shafts and their enclosed soil mass acting as a block foundation) failure. The minimum spacing must include proper allowances for cantilever tolerance of alignment and possible oversizing of the drilled hole.

Drilled Shaft Settlements: A detailed settlement analysis is beyond the scope of this investigation. Based on the soil conditions encountered, we estimate that straight sided drilled shafts designed and constructed as recommended will experience total settlements within 1 inch.

Drilled Shaft Construction: Drilled shaft foundations should be constructed in accordance with Section 02465 of the latest edition of the COHSCS. Based on Borings B-4 and B-6, and depending on the total drilled shaft depth, the drilled shaft excavations will encounter groundwater and water bearing sand/silt, which could cause sidewall sloughing or caving. **To prevent caving and sloughing, AEC recommends that a temporary steel casing (minimum of 5 feet long) and bentonite slurry (if needed) be used to maintain integrity of the shaft excavations.** As indicated in Section 02465 3.02.D of the latest edition of the COHSCS, 4 to 8 percent bentonite should be used for slurry construction; polymer slurry should not be used.

For slurry method, the bentonite slurry should be used prior to encountering ground water or granular soils and the slurry head should be maintained at least 5 feet higher than the ground water at the site during construction. The concrete should be placed using a tremie to displace the lower density slurry. Care must be taken to ensure that tremie is positioned and maintained at the bottom of excavation until a height of 5 feet of concrete has been poured. As more concrete is added, the tremie should be maintained at a minimum distance of 5 feet below the top of the concrete pour.

New drilled shafts should not be excavated within 3 shaft diameters (edge to edge) of an open shaft excavation, or one in which concrete has been placed in the preceding 24 hours, to prevent movement of fresh concrete from the recently filled footing to an adjacent unfilled footing. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the state of stress and possible sloughing in the foundation soils. No shafts should be left open overnight or poured without the prior approval of the Owner's Representative.

In addition, each footing excavation should be inspected by a qualified Owner's Representative prior to placing concrete, to check that (1) the footing excavation has been constructed to the specified dimensions at the recommended depth and formation; and (2) excessive cuttings and any soft-compressible materials have been removed from the bottom of the excavation.

5.2.2.2 Floor Slab

Expansive clays were encountered in Borings B-5 and B-6 at the proposed Horizontal Tank and Two Vertical Tanks. Expansive clays exhibit a high potential to shrink and swell with changes in their moisture contents. The changes in the soil moisture content are usually caused by variations in the seasonal amount of rainfall and evaporation rates or other localized factors like the moisture withdrawal by nearby trees. The seasonal moisture active zone generally extends to about 10 feet below ground in the Greater Houston area, and will be deeper if trees with deep root zones exist adjacent to the structure.

Estimated Soil Movements: Potential Vertical Rise (PVR) is an estimate of the potential of an expansive soil to swell from its current state. For the top 10 feet of the existing soils encountered in Borings B-5 and B-6, the total PVR at the tank sites is estimated to be approximately 1 to 1.2 inches based on in-situ moisture conditions. PVR was computed using the TxDOT test method Tex-124-E.

Additional movements can occur in areas if water is allowed to pond during or after construction on soils with high plasticity, or if highly plastic soils are allowed to dry out prior to fill or concrete placement. High-plasticity clay may also experience shrinkage during periods of dry weather as moisture evaporation occurs at the ground surface and the groundwater table drops. The actual PVR of the site will be highly dependent upon the moisture regime of the soils at the time of construction. Therefore, uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure is critical to reduce potential shrink-swell movement of the floor slab. The PVR can be reduced by replacing the high-plasticity clays with low-expansive compacted select fill or stabilized soils.

Floor Slab: In general, the tolerable differential vertical movement for a common building slab is about 1 inch. A reinforced concrete slab-on-grade can be used for the building floor slab. This option assumes that uniformity and preservation of the moisture contents of the near surface clays during construction and during the life of the structure are maintained adequately, and that any resultant movements can be adequately sustained by the subgrade soils and foundation system.

Subgrade Preparation: Subgrade preparation should extend a minimum of 5 feet beyond the floor slab perimeter. A minimum of 6 inches of surface soils, existing vegetation, trees, roots, and other deleterious materials shall be removed and wasted. The excavation depth should be increased when inspection indicates the presence of organics and deleterious materials to greater depths.

Afterwards, an additional 1.5 feet of existing fill soils (total depth of 2 feet, which includes the 6 inches of surface removal) should be removed. The exposed subgrade should be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications to identify and remove any weak, compressible, or other unsuitable materials; such materials should be replaced with compacted select fill or clean stabilized soils. After proof-rolling, compacted select fill or clean stabilized soils should then be used to achieve the design grades. Select fill or clean on-site stabilized soil should be in accordance with Section 5.3 of this report.

Alternatively, if a moisture barrier underneath the slab is not used, we recommend that the top 6 inches of the exposed subgrade (after surface soils removal of 6 inches or more) be stabilized with either hydrated lime or a combination of hydrated lime and fly ash, depending on the soil conditions. Exposed clay soils (with a PI greater than 10) should be stabilized with a minimum of 5 percent hydrated lime by dry soil weight; whereas sands, silts, and sandy lean clays (with a PI of 10 or less) should be stabilized with at least 3 percent hydrated lime and 7 percent fly ash by dry soil weight. Lime and lime/fly-ash stabilization shall be performed in accordance with Sections 02336 and 02337 of the latest edition of the COHSCS, respectively. The percentage of lime and lime/fly-ash required for stabilization are preliminary estimates for planning purposes only; laboratory testing should be performed to determine optimum contents for stabilization prior to construction. The stabilized soils should be compacted to 95 percent of their ASTM D 698 (Standard Proctor) dry density at a moisture content ranging from optimum to 3 percent above optimum.

Grade Beams: We recommend that foundation grade beams be founded at least 24 inches below the lowest finished grade. The grade beams can be constructed on 6 inch carton forms. If carton forms are used, care should be taken so that the carton forms do not collapse during concrete placement and will not be exposed to water in the grade beam excavations. Surface water should not be allowed to seep into and remain in the carton form space during the life of the structures. If no carton forms will be used, we recommend that tensile reinforcement be placed in both top and bottom of the beams. The drilled shafts and beams should be tied together.

Floor slabs are typically structurally tied to the grade beams. Alternatively, isolating the floor slabs from grade beams with a flexible impervious compound will be beneficial to reduce the potential for slab cracking due to differential soil movement; however, its use will not mitigate the total and differential PVR movements and the floor slabs are expected to move corresponding to the subgrade soils.

5.3 Select Fill

Select fill should consist of uniform, non-active inorganic lean clays with a PI between 10 and 20 percent, and more than 50 percent passing a No. 200 sieve. Excavated material delivered to the site for use as select fill shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. Prior to construction, the Contractor should determine if he or she can obtain qualified select fill meeting the above select fill criteria.

As an alternative to imported fill, on-site soils excavated during construction can be stabilized with either hydrated lime or a combination of hydrated lime and fly ash, depending on the soil conditions. Excavated clay soils (with a PI greater than 10) should be stabilized with a minimum of 5 percent hydrated lime by dry soil weight; whereas sands, silts, and sandy lean clays (with a PI of 10 or less) should be stabilized with at least 3 percent hydrated lime and 7 percent fly ash by dry soil weight. Lime and lime/fly-ash stabilization shall be performed in accordance with Sections 02336 and 02337 of the latest edition of the COHSCS, respectively. The percentage of lime and lime/fly-ash required for stabilization are preliminary estimates for planning purposes only; laboratory testing should be performed to determine optimum contents for stabilization prior to construction. AEC prefers using stabilized on-site clay as select fill since compacted lime-stabilized clay generally has high shear strength, low compressibility, and relatively low permeability. Blended or mixed soils (sand and clay) should not be used as select fill.

All material intended for use as select fill should be tested prior to use to confirm that it meets select fill criteria. The fill should be placed in loose lifts not exceeding 8 inches in thickness. Backfill within 3 feet of walls or columns should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors. The stabilized onsite soils or select fill should be compacted to a minimum of 95 percent of the ASTM D 698 (Standard Proctor) maximum dry unit weight at a moisture content ranging between optimum and 3 percent above optimum.

If imported select fill will be used, at least one Atterberg Limits and one percent passing a No. 200 sieve test shall be performed for each 5,000 square feet (sf) of placed fill, per lift (with a minimum of one set of tests per lift), to determine whether it meets select fill requirements.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

To mitigate site problems that may develop following prolonged periods of rainfall, it is essential to have adequate drainage to maintain a relatively dry and firm surface prior to starting any work at the site. Adequate drainage should be maintained throughout the construction period. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps.

6.2 Groundwater Control

The need for groundwater control will depend on the depth of excavation relative to the groundwater depth at the time of construction. In the event that there is heavy rain prior to or during construction, the groundwater table may be higher than indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas, requiring further evaluation and consideration of the excess hydrostatic pressures.

The Contractor should be responsible for selecting, designing, constructing, maintaining and monitoring a groundwater control system and adapt his operations to ensure the stability of the excavations. Groundwater information presented in Section 4.1 and elsewhere in this report, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.

In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

Groundwater for excavations within saturated sands can be controlled by the installation of wellpoints. The practical maximum dewatering depth for well points is about 15 feet. When groundwater control is required below 15 feet, multiple staged wellpoint or deep wells with submersible pumps have generally proved successful. Generally, the groundwater depth should be lowered at least 3 feet below the excavation bottom.

Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity; the Contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation. We recommend that the Contractor verify the groundwater depths and seepage rates prior to and during construction and retain the services of a dewatering expert (if necessary) to assist him in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

For open cut construction in cohesive soils, the possibility of bottom heave must be considered due to the removal of the weight of excavated soil. In lean and fat clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In silty clays, heave does not typically occur unless an artificially large head of water is created through the use of impervious sheeting in bracing the cut. Guidelines for evaluating bottom stability are presented in Section 5.1.1 of this report.

6.3 Construction Monitoring

Excavation, bedding, and backfilling of underground utilities should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

6.4 Monitoring of Existing Structures

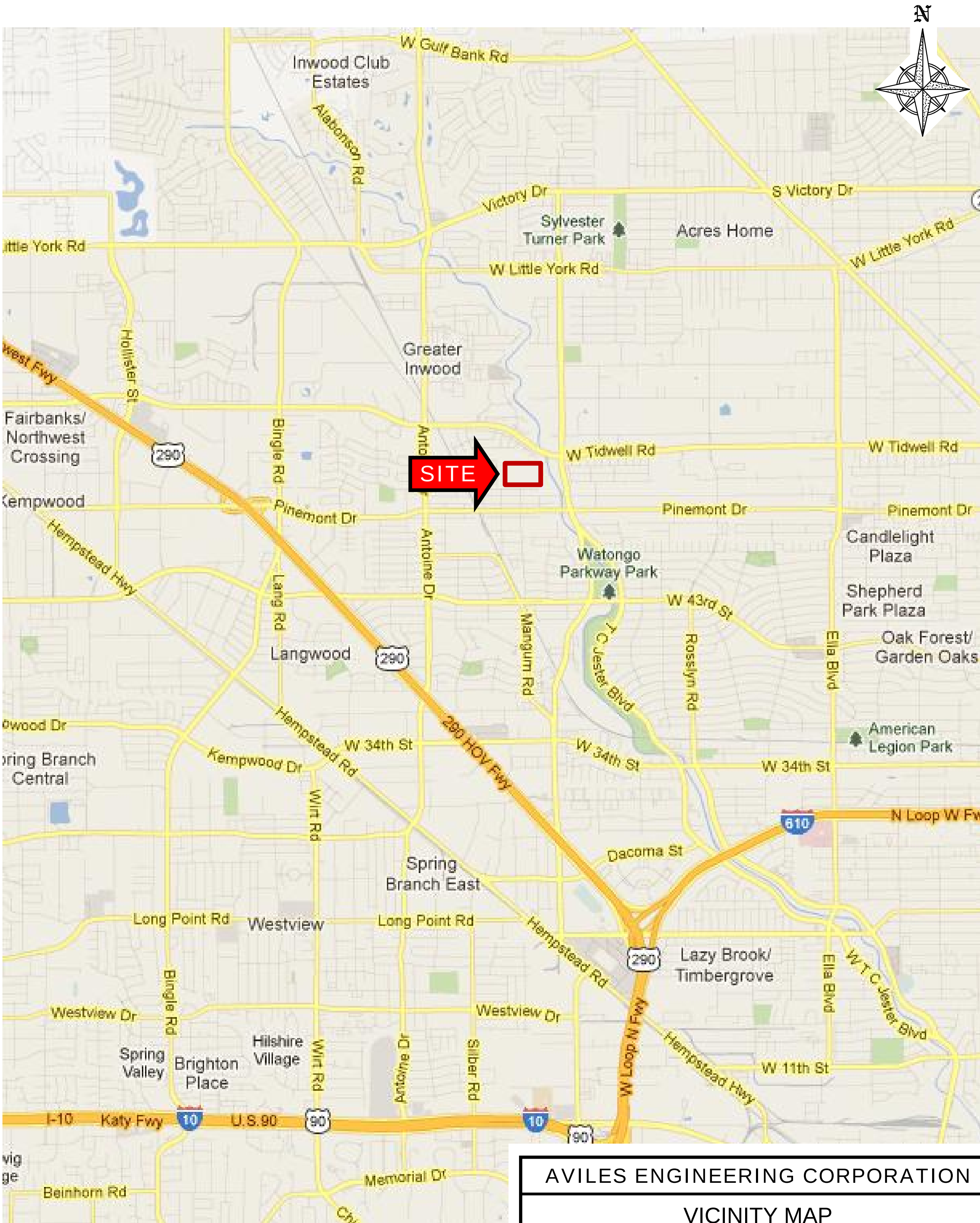
Existing structures in the vicinity of the proposed improvements should be closely monitored prior to, during, and for a period after excavation. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel experience and supervision) may impact ground movement in the vicinity of the excavation. We therefore recommend that the Contractor be required to survey and adequately document the condition of existing structures in the vicinity of the proposed excavation.

7.0 LIMITATIONS

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the dates of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. If conditions encountered during construction are significantly different from those presented in this report; AEC should be notified immediately.

This investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. This report is intended to be used in its entirety. The report has been prepared exclusively for the project and location described in this report. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report, and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures or similar structures located elsewhere, without additional evaluation and/or investigation.

APPENDIX A

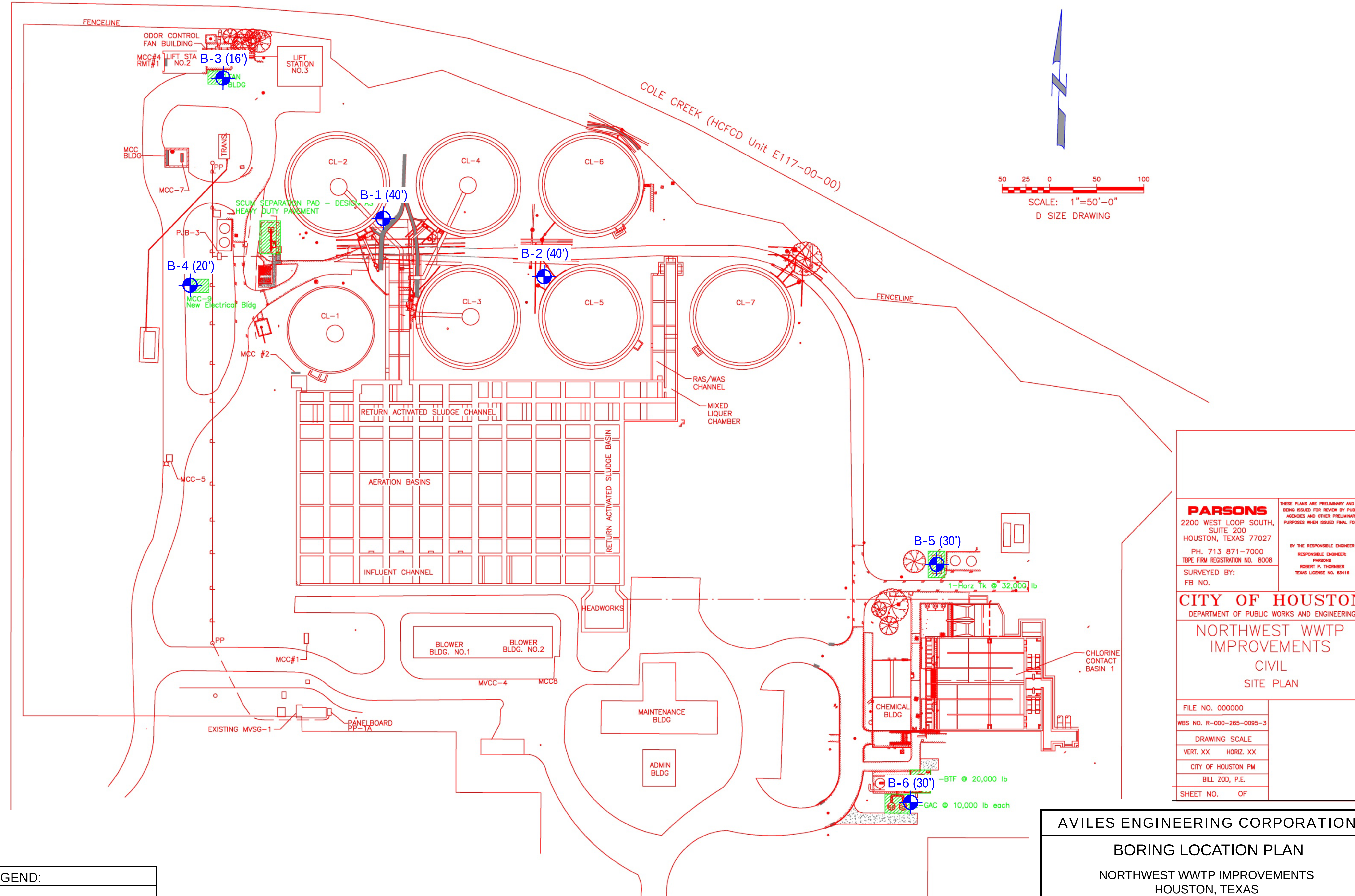


AVILES ENGINEERING CORPORATION

VICINITY MAP

NORTHWEST WWTP IMPROVEMENTS
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	
G178-13	01-29-2014	
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
N.T.S.	BpJ	PLATE A-1



LEGEND:

B-# (X')	*BORING NO. AND (DEPTH IN FEET)*
	BORING LOCATION

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

NORTHWEST WWTP IMPROVEMENTS
HOUSTON, TEXAS

AEC JOB NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G178-13	12-29-13	PARSONS
APPROX. SCALE: 1" = 100'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2

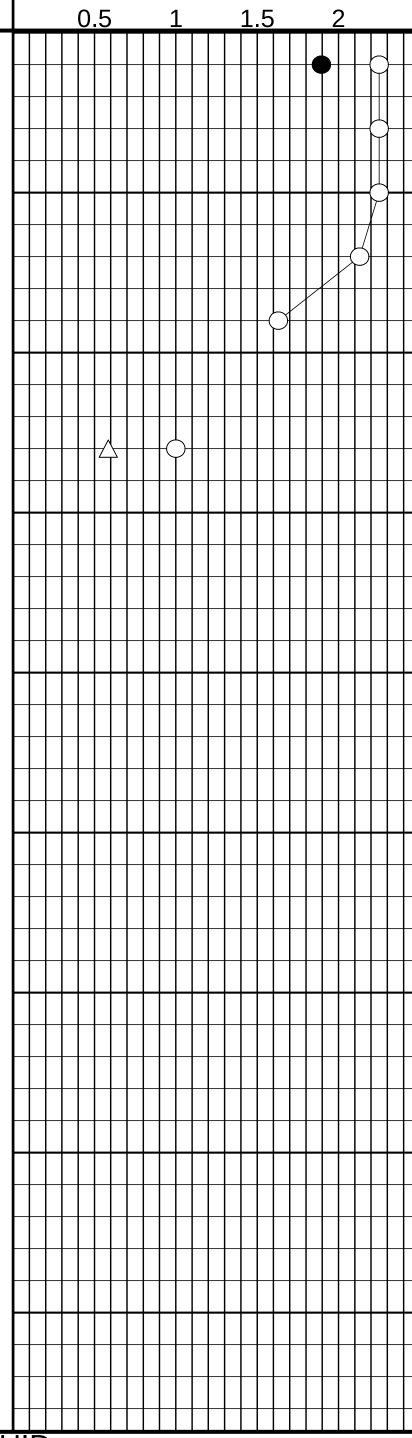
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Northwest WWTP Improvements** BORING **B-1**

DATE **12/18/13** TYPE **4" Dry Auger/Wet Rotary** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3089619.8537
Northing: 13873235.4905
Elevation: 73.72

SHEAR STRENGTH, TSF
△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: light gray and tan Clayey Sand (SC) -with roots 0'-2'		10	125				
	-with shell, calcareous nodules, and brick pieces 2'-4'		5		42	31	13	18
	-with ferrous nodules 4'-6'		7					
6	-with roots 6'-8', and sand seams 6'-10'		18					
			14					
	-light gray, with gravel 10'-12'	13	15		39	27	11	16
12	-with calcareous nodules 12'-14'		16	114				
	-with sand seams 14'-15'		16			38	16	22
	Medium dense, light gray and tan Poorly Graded Sand w/Silt (SP-SM), with clay seams		4		9			
18		21	11					
		14	17					
	-borehole caved in at 20.8' during drilling							
24	Medium dense, tan Poorly Graded Sand (SP), with clay seams, wet	12	25		3			
	-borehole caved in at 26' after 24 hours							
	Medium dense, tan and light gray Silty Sand (SM), with clay seams, wet							
30		18	24					
		15	24					
36								
		21	27		20			
42	Termination depth = 40 feet. Water level at 20.8' after 1/4 hr							

BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **22** FEET WHILE DRILLING
WATER LEVEL AT **20.3** FEET AFTER **24 HRS**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BPJ**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Northwest WWTP Improvements** BORING **B-2**

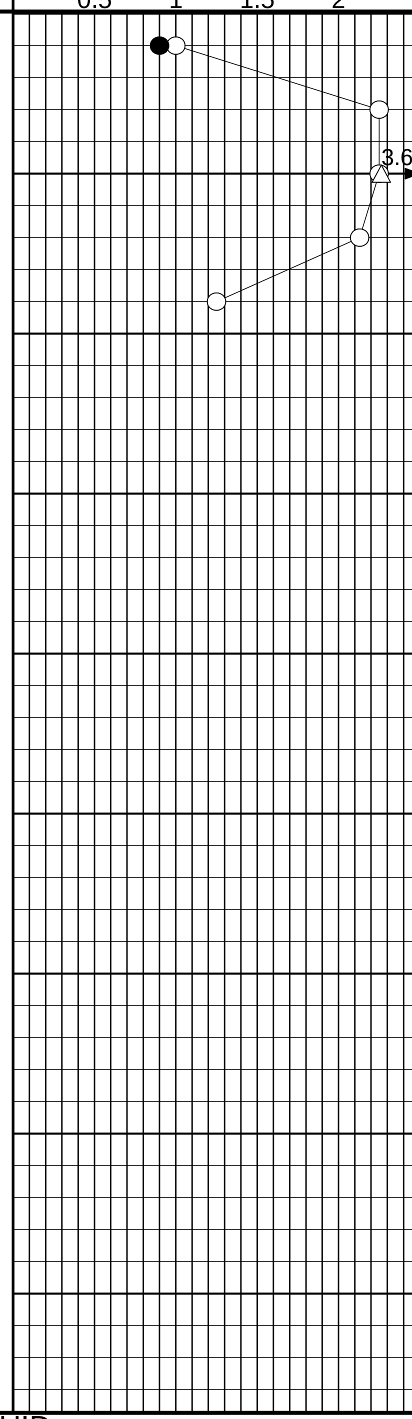
DATE **12/18/13** TYPE **4" Dry Auger/Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

Survey Coordinates (ft):
Easting: 3089773.5396
Northing: 13873165.1211
Elevation: 72.86

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: light gray and tan Clayey Sand (SC) -with roots, sand seams, and gravel 0'-2', and with shell 0'-4'		17	113				
	-with ferrous nodules 2'-4', and calcareous nodules 2'-8'		9		43	37	14	23
	-with siltstone fragments 4'-6'		11	118				
6	-with sand seams 6'-8'		11					
			14		34	26	17	9
12	Fill: loose, light gray and tan Clayey Sand (SC)	7	13					
		5	18					
			24		30	26	18	8
18	Very loose to medium dense, light gray and tan Poorly Graded Sand w/Silt (SP-SM)	4	7					
		3	9		9			
	-borehole caved in at 21.5' during drilling							
24	-wet at 23'	10	24					
30	-light gray 28'-40'	17	24					
36		14	25					
	-with clayey sand seams 38'-40'	26	25		5			
42	Termination depth = 40 feet. Water level at 21.5' after 1/4 HR							

BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **23** FEET WHILE DRILLING
WATER LEVEL AT **18.8** FEET AFTER **24 HRS**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BPJ**

PROJECT: **Northwest WWTP Improvements**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-3**

DATE **12/18/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

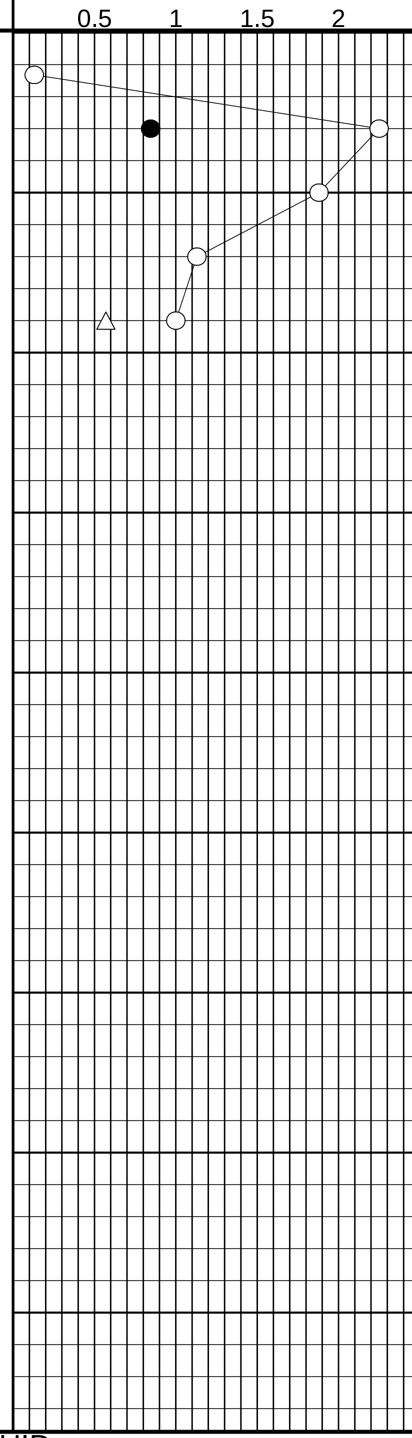
DESCRIPTION

Survey Coordinates (ft):
Easting: 3089424.3406
Northing: 13873364.6306
Elevation: 69.02

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 7.75" Concrete							
	Fill: light gray and tan stabilzed Clayey Sand (SC), with shell -with stabilized sand 0.5'-1', calcareous nodules, and siltstone fragments 1'-2' -with gravel and crushed base 2'-4'		18					
					24	27	19	8
			18	110				
			11					
6	Fill: gray Silty Clayey Sand (SC-SM) -with sand seams 4'-6' and shell 4'-8' -with sand seams 8'-10'				34	24	18	6
			16					
			14	114				
	Very loose to loose, gray Silty Clayey Sand (SC-SM)	3	15					
12					23	24	17	7
		6	21					
		5	17					
	Termination depth = 16 feet.							
18								
24								
30								
36								
42								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **16** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **N/A** FEET WHILE DRILLING

WATER LEVEL AT **N/A** FEET AFTER **COMPLETE**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BPJ**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Northwest WWTP Improvements** BORING **B-4**

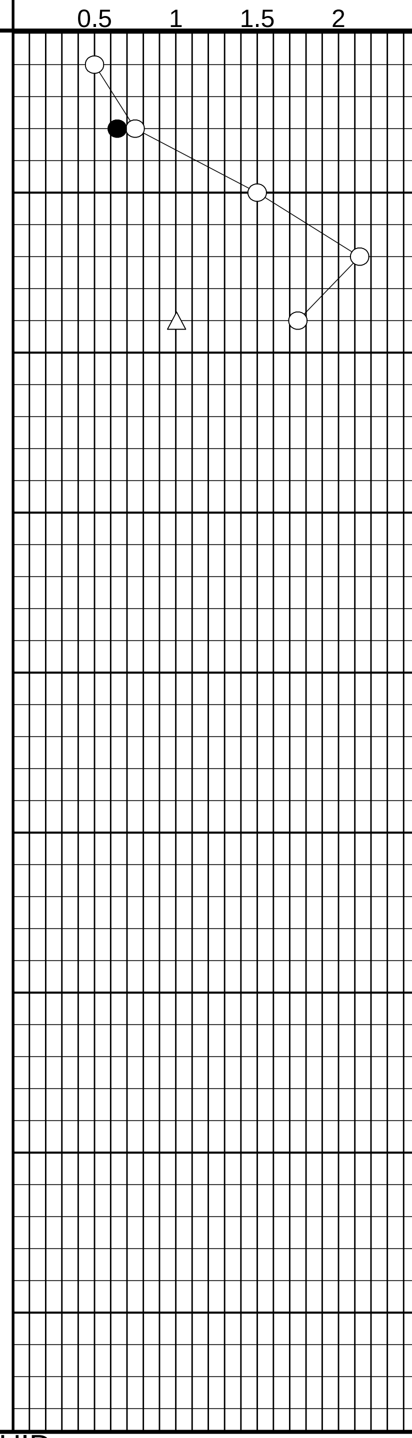
DATE **12/18/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3089399.7245
Northing: 13873157.8167
Elevation: 73.32

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: gray and tan Sandy Silt (ML), with clay pockets and sand seams		11		68	19	16	3
	Fill: stiff to very stiff, light gray and tan Sandy Lean Clay (CL) -with crushed base and siltstone fragments 2'-4'		20	111				
			18					
6	Very stiff to hard, tan and light gray Sandy Lean Clay (CL) -with sand pockets 6'-8' -with gravel 8'-10'		18		53	37	16	21
			18	110				
	Loose to medium dense, light gray Clayey Sand (SC), with silty sand seams	8	14		44			
12		8	15					
		13	24					
	Loose to medium dense, tan Poorly Graded Sand w/Silt (SP-SM)	18	19		11			
18		10	24					
	Termination depth = 20 feet.							
24								
30								
36								
42								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **N/A** FEET WHILE DRILLING
WATER LEVEL AT **N/A** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BPJ**

PROJECT NO. G178-13 **PLATE A-6**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Northwest WWTP Improvements** BORING **B-5**

DATE **12/19/13** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3090196.6926
Northing: 13872880.0302
Elevation: 73.17

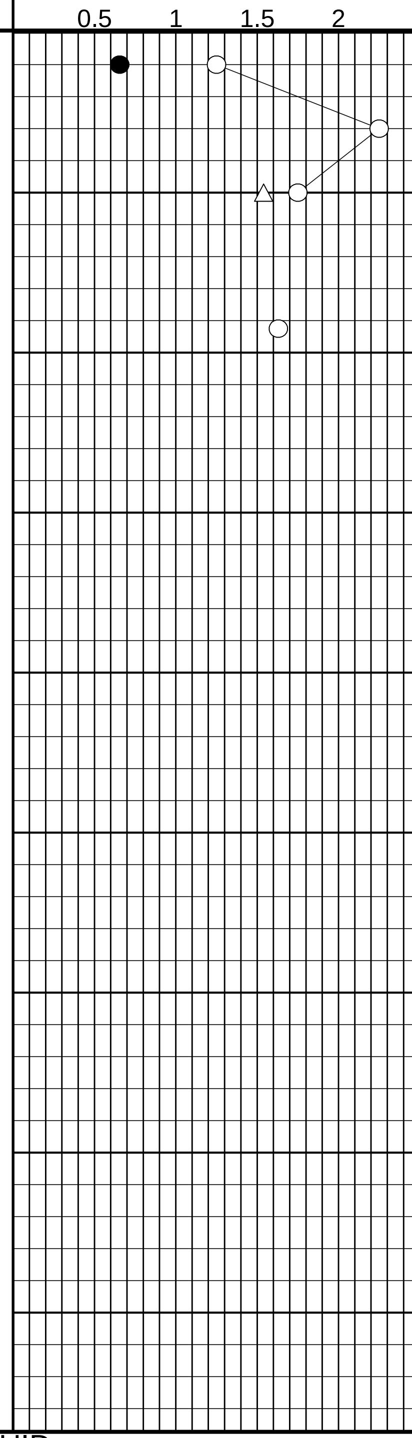
DESCRIPTION

- Fill: stiff to hard, tan and light gray Sandy Lean Clay (CL)
  - -with roots and calcareous nodules 0'-2', and ferrous nodules 0'-4'
  - -with sand pockets and siltstone fragments 2'-4'
  - -with calcareous nodules and fat clay pockets 4'-6'
- Very stiff, light gray and tan Sandy Lean Clay (CL)
  - -with sand seams 6'-8'
  - -with calcareous nodules and siltstone fragments 8'-10'
  - -with ferrous nodules 10'-12'
- Medium dense to dense, light gray Silty Sand (SM)
  - -borehole caved in at 22.4' during drilling
  - -wet at 23'
- Termination depth = 30 feet.

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	20	106				
	12		53	35	14	21
	16	111				
	16					
25	18		57	41	14	27
19	17					
18	9		14			
20	10					
22	10					
18	8		12			
40	22					
23	23					

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



DEPTH IN FEET: 0, 6, 12, 18, 24, 30, 36, 42

BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **23** FEET WHILE DRILLING
WATER LEVEL AT **22.4** FEET AFTER **10 MIN**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BPJ**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

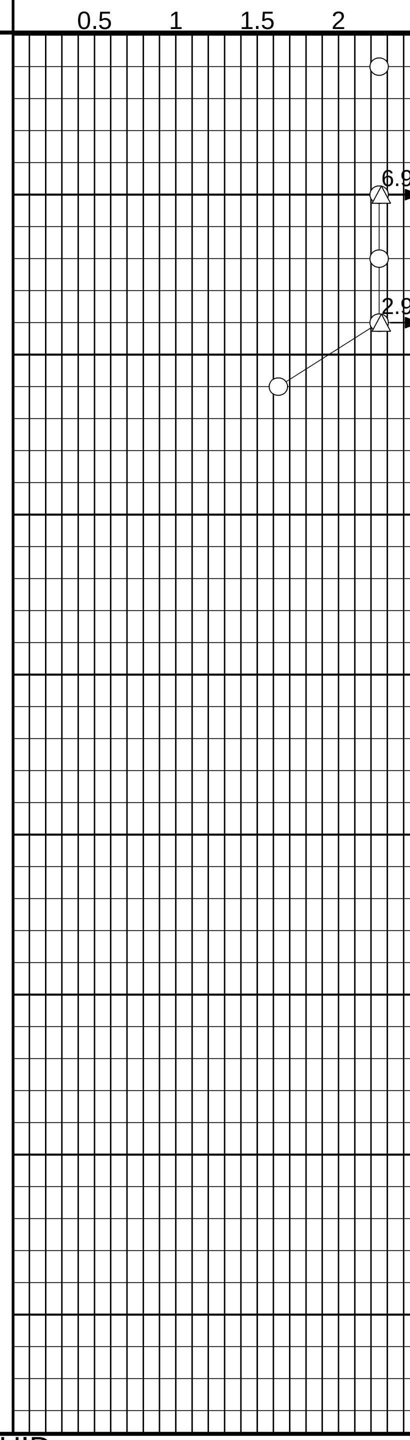
PROJECT: **Northwest WWTP Improvements** BORING **B-6**

DATE **12/19/13** TYPE **4" Dry Auger/Wet Rotary** LOCATION **See Boring Location Plan**

Survey Coordinates (ft):
Easting: 3090192.0345
Northing: 13872625.1005
Elevation: 73.59

SHEAR STRENGTH, TSF: △ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: hard, tan and light gray Sandy Lean Clay (CL), with roots, shell, and ferrous nodules		13		53	37	15	22
	Very dense, tan and gray Clayey Sand (SC), with roots	50/5"	5		48			
	Hard, tan and gray Sandy Lean Clay (CL), with ferrous nodules		15	114				
6	-light gray, tan, and red 6'-8'		14		63	45	14	31
			12	120				
	Dense, light gray and tan Silty Sand (SM) -with clay seams 10'-12'		10		29	18	6	2
12		48	10					
	Dense, light gray Silt (ML)	32	8		92			
	Dense to very dense, light gray Poorly Graded Sand w/Silt (SP-SM)	37	5					
18		47	5					
24	-borehole caved in at 23' during drilling	37	4		5			
30	-tan and gray 28'-30'	50/4"	12					
	Termination depth = 30							
36								
42								



BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **23** FEET WHILE DRILLING
WATER LEVEL AT **N/A** FEET AFTER **COMPLETE**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BPJ**

KEY TO SYMBOLS

Strata symbols

Symbol	Description
	Fill
	Poorly graded sand with silt
	Poorly graded sand
	Silty sand
	Paving
	Poorly graded clayey silty sand
	Low plasticity clay
	Clayey sand
	Silt

Misc. Symbols

Symbol	Description
	Water table depth during drilling
	Subsequent water table depth
	Pocket Penetrometer
	Unconfined Compression
	Confined Compression

Soil Samplers

Symbol	Description
	Undisturbed thin wall Shelby tube
	Standard penetration test
	Rock core

ENGINEERING CORP.

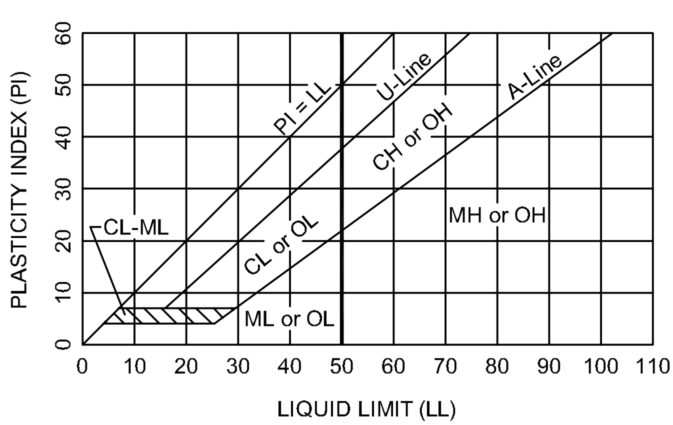
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)		SILTS AND CLAYS (Liquid Limit Less Than 50%)		ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
		SILTS AND CLAYS (Liquid Limit 50% or More)		MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS

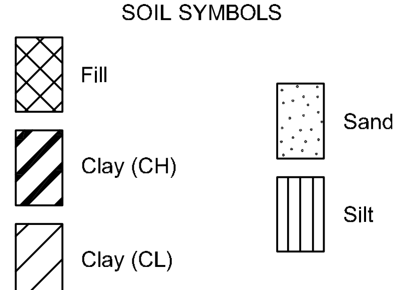


PLATE A-10

AVILES ENGINEERING CORP.

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

	6"		3"		3/4"		#4		#10		#40		#200		
BOULDERS		COBBLES		GRAVEL: COARSE		GRAVEL: FINE		SAND: COARSE		SAND: MEDIUM		SAND: FINE		SILT	CLAY
	152		76.2		19.1		4.76		2.00		0.420		0.074		0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose	<4 bpf
Loose	5-10 bpf
Medium Dense	11-30 bpf
Dense	31-50 bpf
Very Dense	>50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part 1)
Hydrometer Analysis	D 422	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
Shrinkage Limit	D 427	Tex-107-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
Permeability (constant head)	D 2434	-
Consolidation	D 2435	-
Direct Shear	D 3080	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Pinhole Test	D 4647	-
California Bearing Ratio	D 1883	-
Unified Soil Classification System	D 2487	Tex-142-E

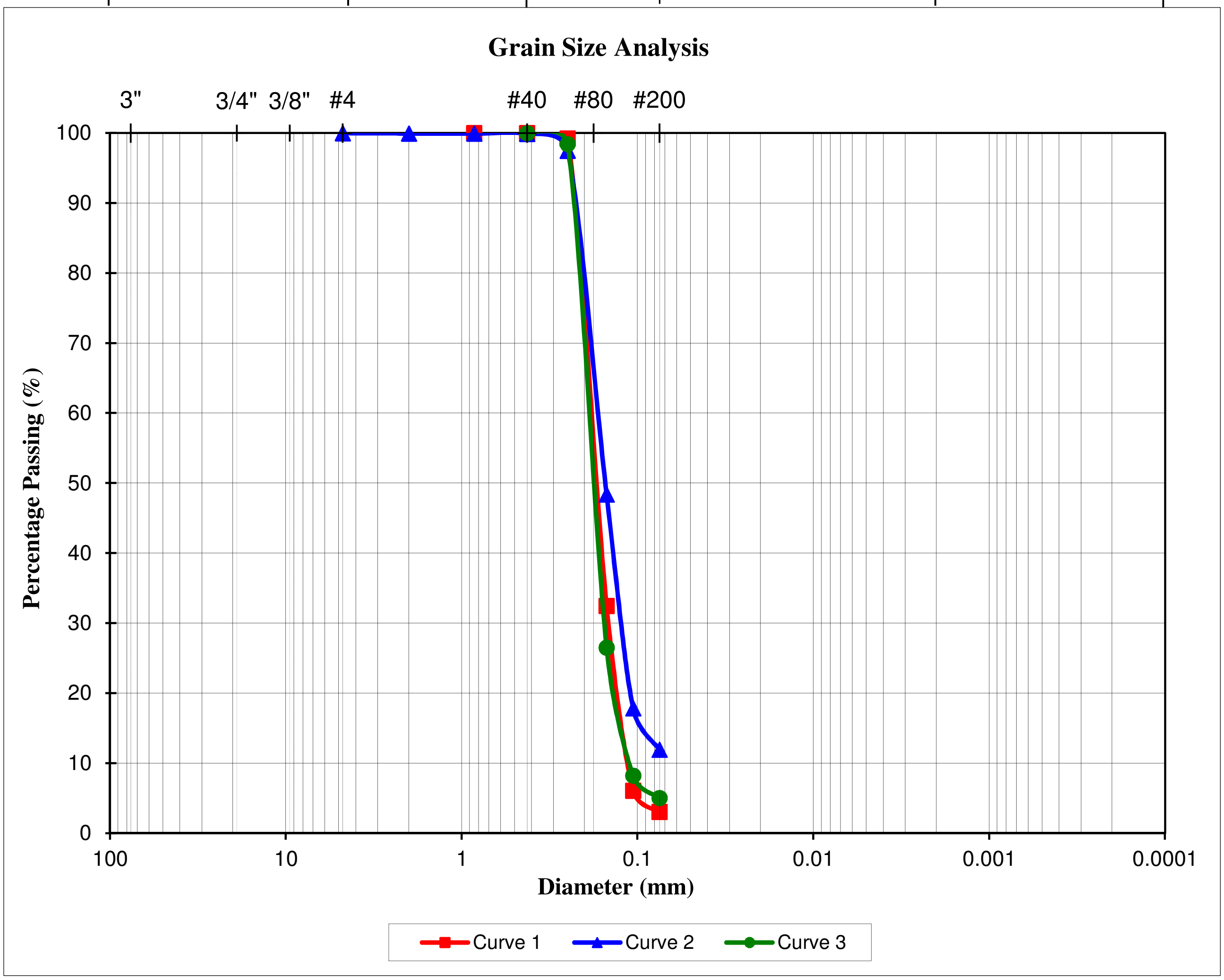
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE

Project : Northwest WWTP Improvements **Job No.:** G178-13
Location of Project: Houston, Texas **Date of Testing:**

	Gravel	Sand: Coarse to Medium	Sand: Fine	Silt	Clay	



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc
1	B-1	23-25	Poorly Graded Sand (SP)	1.66	1.02
2	B-5	18-20	Silty Sand (SM)	N/A	N/A
3	B-6	23-25	Poorly Graded Sand W/Silt (SP-SM)	1.74	1.13

SUMMARY OF SOIL TEST RESULTS
NORTHWEST WASTEWATER TREATMENT PLANT IMPROVEMENTS
HOUSTON, TEXAS

BORING NO.	SAMPLE				SPT (blows/ft)	WATER CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PERCENT PASSING SIEVE NO. 200 (%)	SHEAR STRENGTH (tsf)			USCS GROUP NAME
	NO.	DEPTH (ft)		TYPE				LL (%)	PL (%)	PI (%)		UNCONFINED COMPRESSION TEST	UU TEST	POCKET PENETRO-METER (TORVANE)	
		TOP	BOTTOM												
B-1	1	0	2	UD		10	125					1.90		2.25	Fill: Clayey Sand (SC)
	2	2	4	UD		5		31	13	18	42			2.25	Fill: Clayey Sand (SC)
	3	4	6	UD		7								2.25	Fill: Clayey Sand (SC)
	4	6	8	UD		18								2.13	Fill: Clayey Sand (SC)
	5	8	10	UD		14								1.63	Fill: Clayey Sand (SC)
	6	10.5	12	SS	13	15		27	11	16	39				Fill: Clayey Sand (SC)
	7	12	14	UD		16	114						0.59	1.00	Fill: Clayey Sand (SC)
	8	14	15	UD		16		38	16	22					Fill: Clayey Sand (SC)
	9	15	16	UD		4					9				Poorly Graded Sand w/Silt (SP-SM)
	10	16.5	18	SS	21	11									Poorly Graded Sand w/Silt (SP-SM)
	11	18.5	20	SS	14	17									Poorly Graded Sand w/Silt (SP-SM)
	12	23.5	25	SS	12	25					3				Poorly Graded Sand (SP)
	13	28.5	30	SS	18	24									Silty Sand (SM)
	14	33.5	35	SS	15	24									Silty Sand (SM)
	15	38.5	40	SS	21	27					20				Silty Sand (SM)
B-2	1	0	2	UD		17	113					0.90		1.00	Fill: Clayey Sand (SC)
	2	2	4	UD		9		37	14	23	43			2.25	Fill: Clayey Sand (SC)
	3	4	6	UD		11	118						3.60	2.25	Fill: Clayey Sand (SC)
	4	6	8	UD		11								2.13	Fill: Clayey Sand (SC)
	5	8	10	UD		14		26	17	9	34			1.25	Fill: Clayey Sand (SC)
	6	10.5	12	SS	7	13									Fill: Clayey Sand (SC)
	7	12.5	12	SS	5	18									Fill: Clayey Sand (SC)
	8	14	16	UD		24		26	18	8	30				Fill: Clayey Sand (SC)
	9	16.5	18	SS	4	7									Poorly Graded Sand w/Silt (SP-SM)
	10	18.5	20	SS	3	9					9				Poorly Graded Sand w/Silt (SP-SM)
	11	23.5	25	SS	10	24									Poorly Graded Sand w/Silt (SP-SM)
	12	28.5	30	SS	17	24									Poorly Graded Sand w/Silt (SP-SM)
	13	33.5	35	SS	14	25									Poorly Graded Sand w/Silt (SP-SM)
	14	38.5	40	SS	26	25					5				Poorly Graded Sand w/Silt (SP-SM)
B-3	1	0	0.645	RC											7.75-in Pavement (PCC)
	2	0.645	2	UD		18								0.13	Fill: Clayey Sand (SC)
	3	2	4	UD		18	110	27	19	8	24	0.85		2.25	Fill: Clayey Sand (SC)
	4	4	6	UD		11								1.88	Fill: Silty Clayey Sand (SC-SM)
	5	6	8	UD		16		24	18	6	34			1.13	Fill: Silty Clayey Sand (SC-SM)
	6	8	10	UD		14	114						0.57	1.00	Fill: Silty Clayey Sand (SC-SM)
	7	10.5	12	SS	3	15									Silty Clayey Sand (SC-SM)
	8	12.5	14	SS	6	21		24	17	7	23				Silty Clayey Sand (SC-SM)
	9	14.5	16	SS	5	17									Silty Clayey Sand (SC-SM)

(continued on next page)

SUMMARY OF SOIL TEST RESULTS
NORTHWEST WASTEWATER TREATMENT PLANT IMPROVEMENTS
HOUSTON, TEXAS

BORING NO.	SAMPLE				SPT (blows/ft)	WATER CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PERCENT PASSING SIEVE NO. 200 (%)	SHEAR STRENGTH (tsf)			USCS GROUP NAME
	NO.	DEPTH (ft)		TYPE				LL (%)	PL (%)	PI (%)		UNCONFINED COMPRESSION TEST	UU TEST	POCKET PENETRO-METER (TORVANE)	
		TOP	BOTTOM												
B-4	1	0	2	UD		11		19	16	3	68			0.50	Fill: Sandy Silt (ML)
	2	2	4	UD		20	111					0.64		0.75	Fill: Sandy Lean Clay (CL)
	3	4	6	UD		18								1.50	Fill: Sandy Lean Clay (CL)
	4	6	8	UD		18		37	16	21	53			2.13	Sandy Lean Clay (CL)
	5	8	10	UD		18	110						1.01	1.75	Sandy Lean Clay (CL)
	6	10.5	12	SS	8	14					44				Clayey Sand (SC)
	7	12.5	14	SS	8	15									Clayey Sand (SC)
	8	14.5	16	SS	13	24									Clayey Sand (SC)
	9	16.5	18	SS	18	19					11				Poorly Graded Sand w/Silt (SP-SM)
	10	18.5	20	SS	10	24									Poorly Graded Sand w/Silt (SP-SM)
B-5	1	0	2	UD		20	106					0.66		1.25	Fill: Sandy Lean Clay (CL)
	2	2	4	UD		12		35	14	21	53			2.25	Fill: Sandy Lean Clay (CL)
	3	4	6	UD		16	111						1.54	1.75	Fill: Sandy Lean Clay (CL)
	4	6	8	UD		16									Sandy Lean Clay (CL)
	5	8.5	10	SS	25	18		41	14	27	57			1.63	Sandy Lean Clay (CL)
	6	10.5	12	SS	19	17									Sandy Lean Clay (CL)
	7	12.5	14	SS	18	9					14				Silty Sand (SM)
	8	14.5	16	SS	20	10									Silty Sand (SM)
	9	16.5	18	SS	22	10									Silty Sand (SM)
	10	18.5	20	SS	18	8					12				Silty Sand (SM)
	11	23.5	25	SS	40	22									Silty Sand (SM)
	12	28.5	30	SS	23	23									Silty Sand (SM)
B-6	1	0	2	UD		13		37	15	22	53			2.25	Fill: Sandy Lean Clay (CL)
	2	2.5	4	SS	50/5"	5					48				Clayey Sand (SC)
	3	4	6	UD		15	114						6.90	2.25	Sandy Lean Clay (CL)
	4	6	8	UD		14		45	14	31	63			2.25	Sandy Lean Clay (CL)
	5	8	10	UD		12	120						2.90	2.25	Sandy Lean Clay (CL)
	6	10	12	UD		10		18	6	2	29			1.63	Silty Sand (SM)
	7	12.5	14	SS	48	10									Silty Sand (SM)
	8	14.5	16	SS	32	8					92				Silt (ML)
	9	16.5	18	SS	37	5									Poorly Graded Sand w/Silt (SP-SM)
	10	18.5	20	SS	47	5									Poorly Graded Sand w/Silt (SP-SM)
	11	23.5	25	SS	37	4					5				Poorly Graded Sand w/Silt (SP-SM)
	12	28.5	30	SS	50/4"	12									Poorly Graded Sand w/Silt (SP-SM)

Notes: (1) UD = Undisturbed Sample (Shelby Tube); SS = Split Spoon Sample; AG = Auger Cuttings; RC = Pavement Core
(2) LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; UU = Unconsolidated-Undrained Test
(3) PCC = Portland Cement Concrete

AVILES
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APPENDIX B

Table 1. Design Soil Parameters for Sheet Piles, Northwest WWTP Improvements (Based on Boring B-1)

Depth (ft)	Elevation (ft)	Soil Type	γ (pcf)	γ' (pcf)	Short-Term				Long-Term			
					C_u (psf)	ϕ_u (deg)	K_a	K_p	C' (psf)	ϕ' (deg)	K_a	K_p
0 to 8	74 to 66	Fill: SC	138	75	500	0	1.00	1.00	50	20	0.49	2.04
8 to 15	66 to 59	Fill: Medium dense SC	132	70	0	28	0.36	2.77	0	28	0.36	2.77
15 to 22	59 to 52	Medium dense SP-SM w/silt	135	73	0	28	0.36	2.77	0	28	0.36	2.77
22 to 40	52 to 34	Medium dense SM	138	75	0	30	0.33	3.00	0	30	0.33	3.00

Notes:
(1) γ = wet unit weight of soil, γ' = buoyant unit weight of soil;
(2) C_u = undrained cohesion, ϕ_u = angle of internal friction, under short term conditions; C' = effective cohesion, ϕ' = effective friction angle, under long term conditions;
(3) Ka = coefficient of active earth pressure for level backfill, Kp = coefficient of passive earth pressure for level backfill: A minimum FS of 2 should be used for passive earth pressure resistance.
(4) SC = clayey sand, SM = silty sand, SP-SM = poorly graded sand with silt.

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Table 2. Design Soil Parameters for Sheet Piles, Northwest WWTP Improvements (Based on Boring B-2)

Depth (ft)	Elevation (ft)	Soil Type	γ (pcf)	γ' (pcf)	Short-Term				Long-Term			
					C_u (psf)	ϕ_u (deg)	K_a	K_p	C' (psf)	ϕ' (deg)	K_a	K_p
0 to 2	73 to 71	Fill: SC	132	70	500	0	1.00	1.00	50	20	0.49	2.04
2 to 8	71 to 65	Fill: SC	131	69	500	0	1.00	1.00	0	26	0.39	2.56
8 to 16	65 to 57	Fill: loose SC	126	64	0	26	0.39	2.56	0	26	0.39	2.56
16 to 22	57 to 51	Very loose SP-SM	120	58	0	25	0.41	2.46	0	25	0.41	2.46
22 to 28	51 to 45	Loose SP-SM	136	74	0	26	0.39	2.56	0	26	0.39	2.56
28 to 40	45 to 33	Medium dense SP-SM	138	75	0	30	0.33	3.00	0	30	0.33	3.00

Notes: (1) γ = wet unit weight of soil, γ' = buoyant unit weight of soil;
(2) C_u = undrained cohesion, ϕ_u = angle of internal friction, under short term conditions; C' = effective cohesion, ϕ' = effective friction angle, under long term conditions;
(3) Ka = coefficient of active earth pressure for level backfill, Kp = coefficient of passive earth pressure for level backfill: A minimum FS of 2 should be used for passive earth pressure resistance.
(4) SC = clayey sand, SM = silty sand, SP-SM = poorly graded sand with silt.

ENGINEERING CORP.

APPENDIX C

Plates C-1a thru C-1c	Straight Sided Drilled Shaft Axial Capacity Curves for Electrical Building and Scum Separation Pad
Plates C-2a thru C-2c	Straight Sided Drilled Shaft Axial Capacity Curves for Horizontal Hydropneumatic Tank Pad
Plates C-3a thru C-3c	Straight Sided Drilled Shaft Axial Capacity Curves for Chemical Storage Tanks Pad

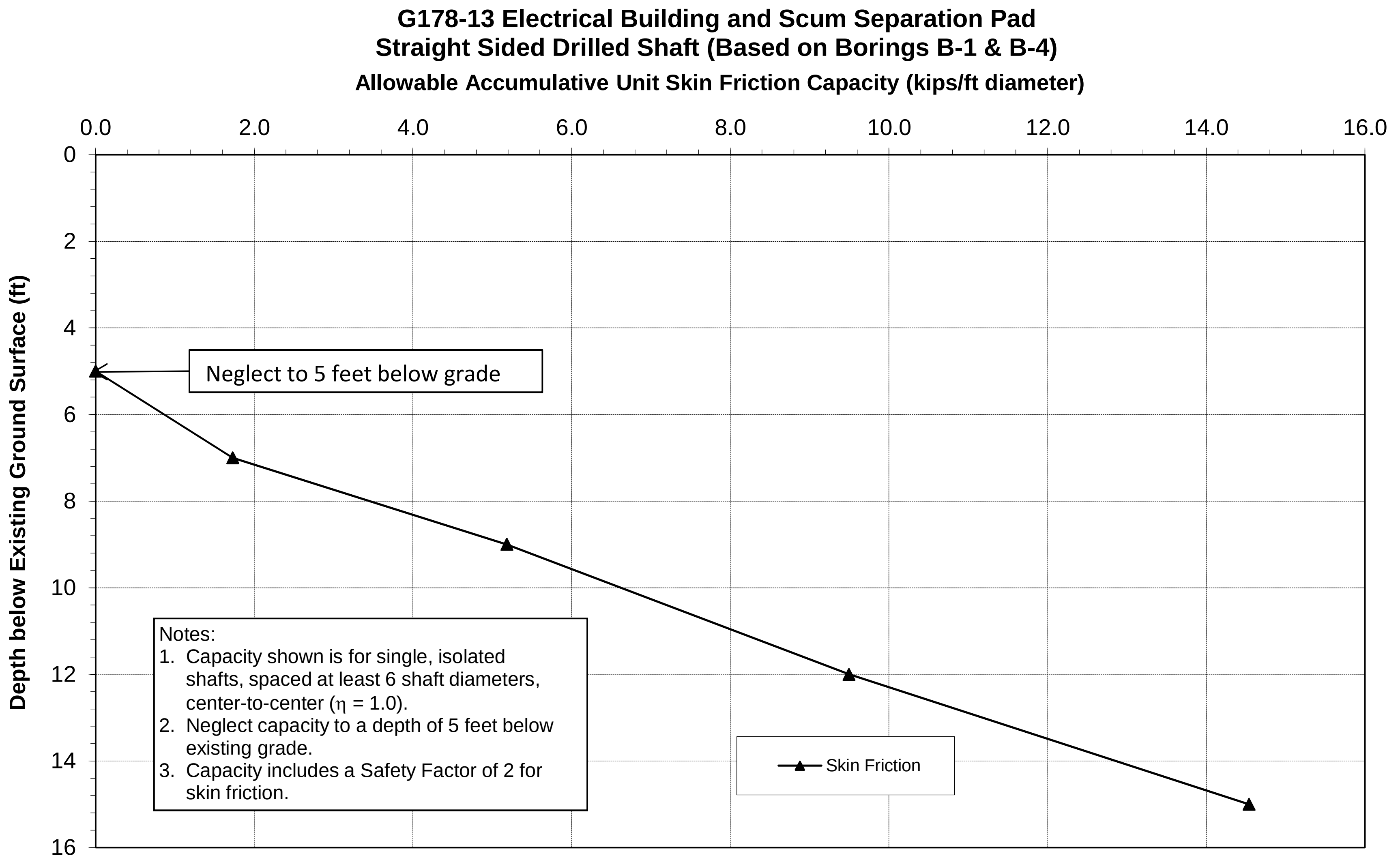
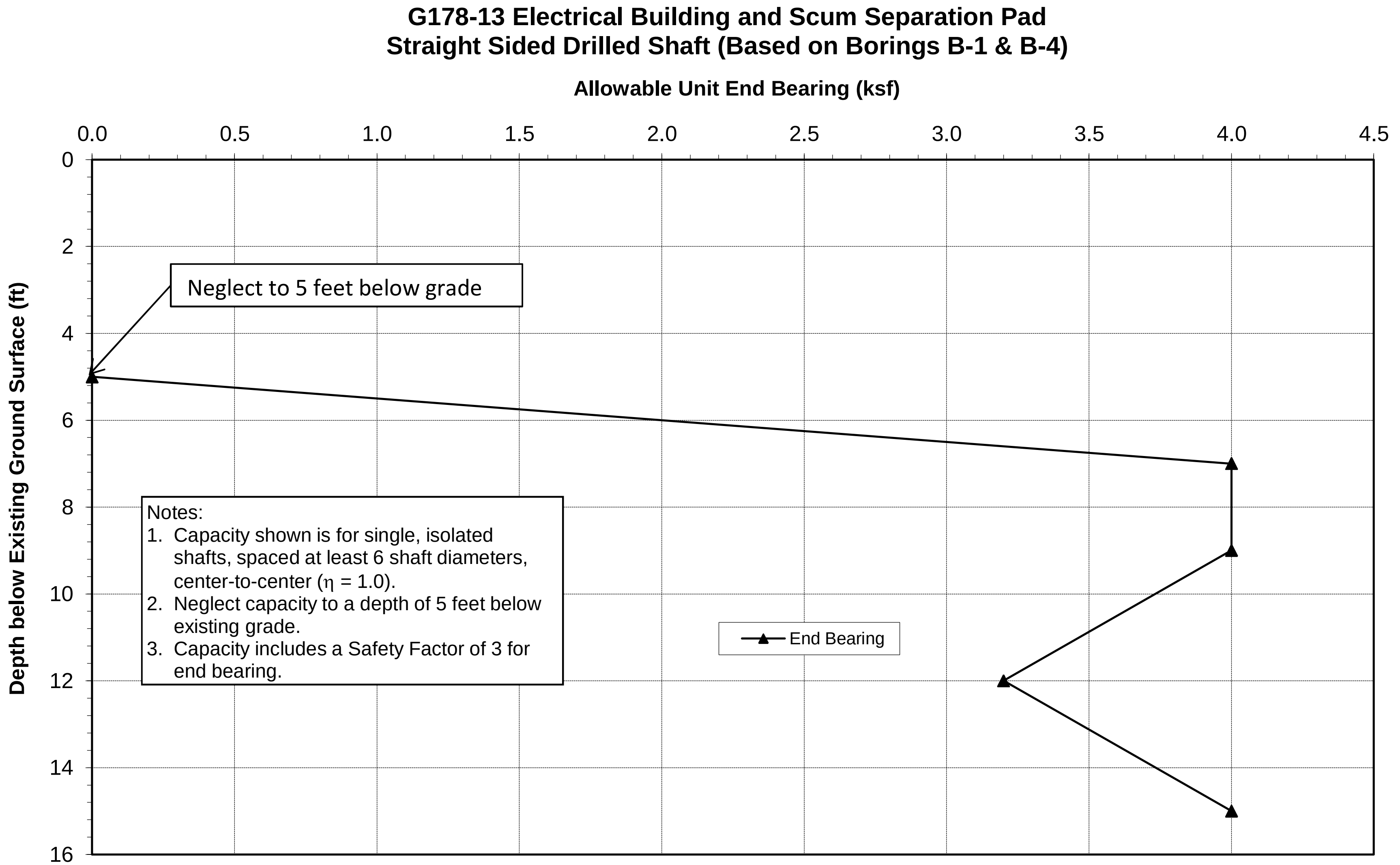
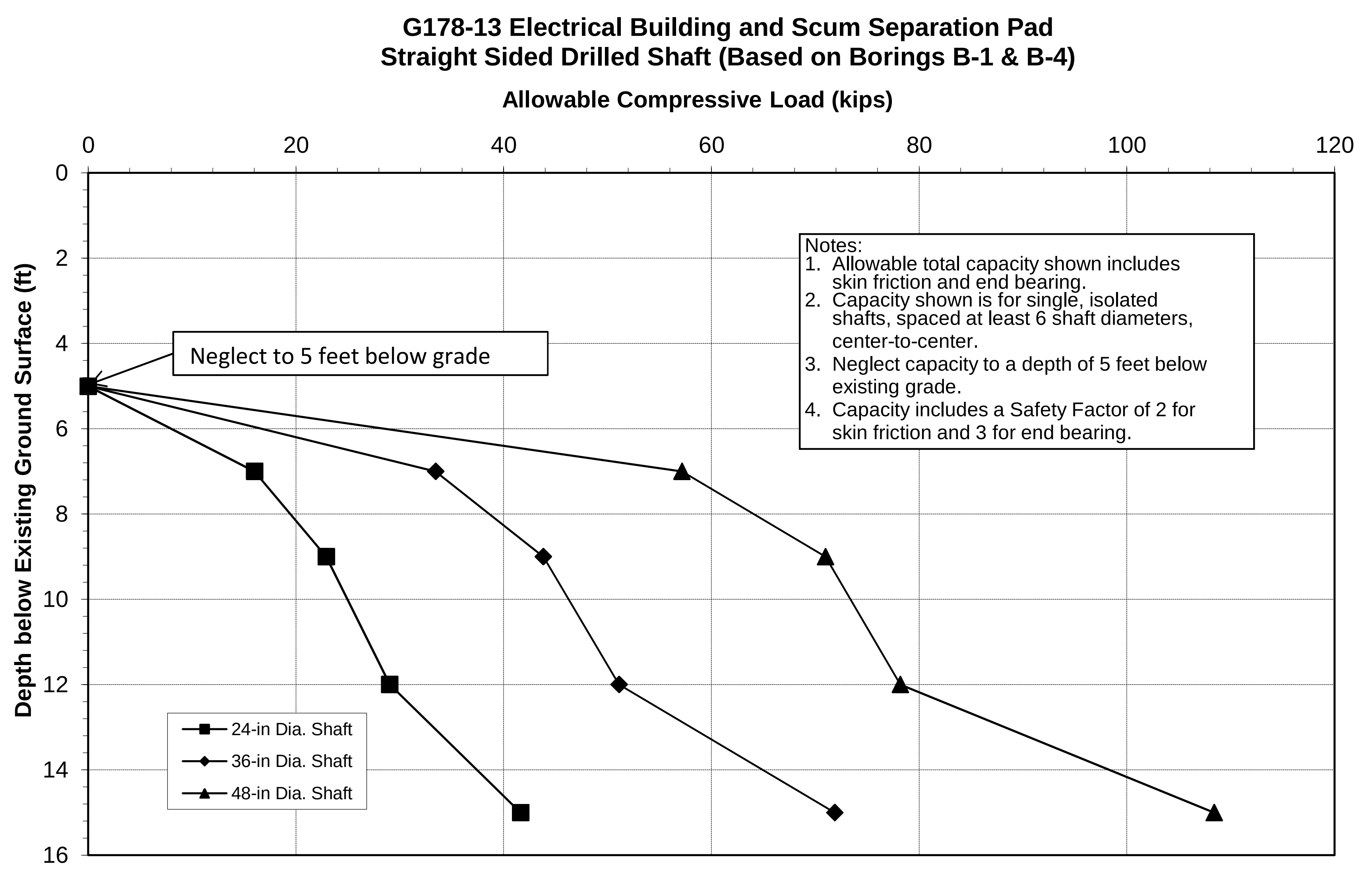

G178-13 Electrical Building and Scum Separation Pad
Straight Sided Drilled Shaft (Based on Borings B-1 & B-4)
Allowable Accumulative Unit Skin Friction Capacity (kips/ft diameter)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
Depth below Existing Ground Surface (ft)
0
2
4
6
8
10
12
14
16
Neglect to 5 feet below grade
Notes:
1. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center (η = 1.0).
2. Neglect capacity to a depth of 5 feet below existing grade.
3. Capacity includes a Safety Factor of 2 for skin friction.
Skin Friction

Plate C-1a



G178-13 Electrical Building and Scum Separation Pad
Straight Sided Drilled Shaft (Based on Borings B-1 & B-4)
Allowable Unit End Bearing (ksf)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Depth below Existing Ground Surface (ft)
0
2
4
6
8
10
12
14
16
Neglect to 5 feet below grade
Notes:
1. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center (η = 1.0).
2. Neglect capacity to a depth of 5 feet below existing grade.
3. Capacity includes a Safety Factor of 3 for end bearing.
End Bearing

Plate C-1b



G178-13 Electrical Building and Scum Separation Pad
Straight Sided Drilled Shaft (Based on Borings B-1 & B-4)
Allowable Compressive Load (kips)
0
20
40
60
80
100
120
Depth below Existing Ground Surface (ft)
0
2
4
6
8
10
12
14
16
Neglect to 5 feet below grade
Notes:
1. Allowable total capacity shown includes skin friction and end bearing.
2. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center.
3. Neglect capacity to a depth of 5 feet below existing grade.
4. Capacity includes a Safety Factor of 2 for skin friction and 3 for end bearing.
24-in Dia. Shaft
36-in Dia. Shaft
48-in Dia. Shaft

Plate C-1c

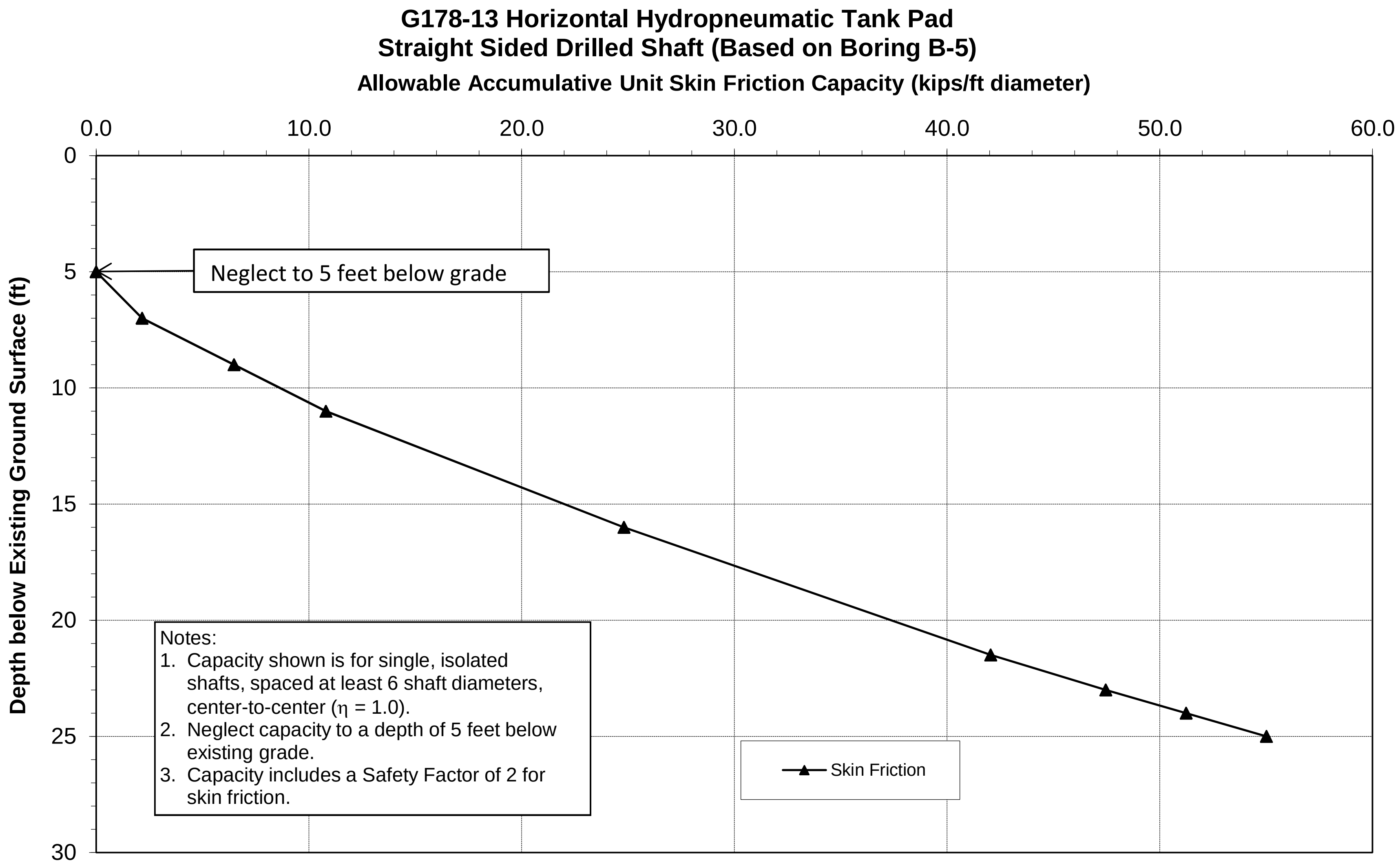
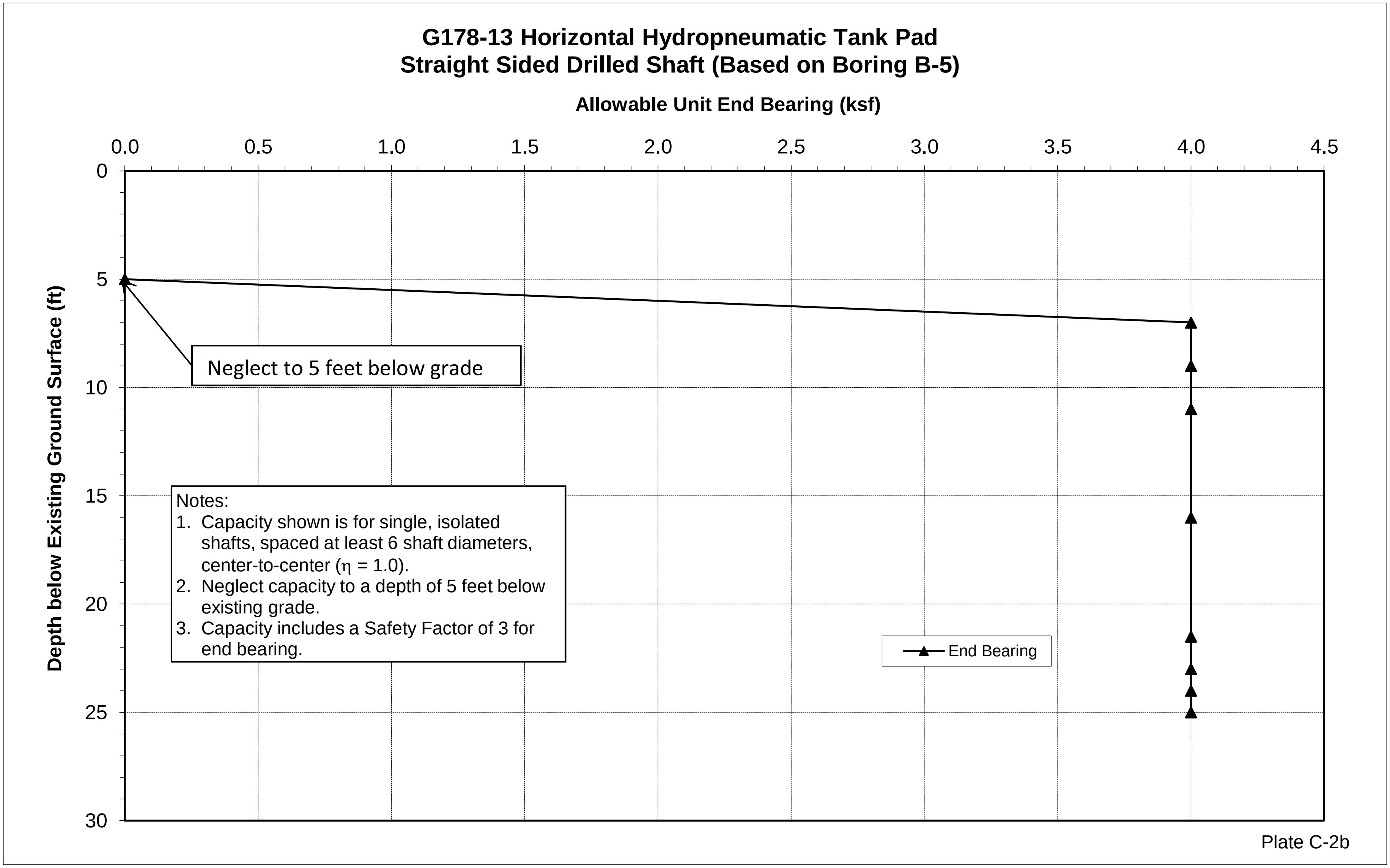
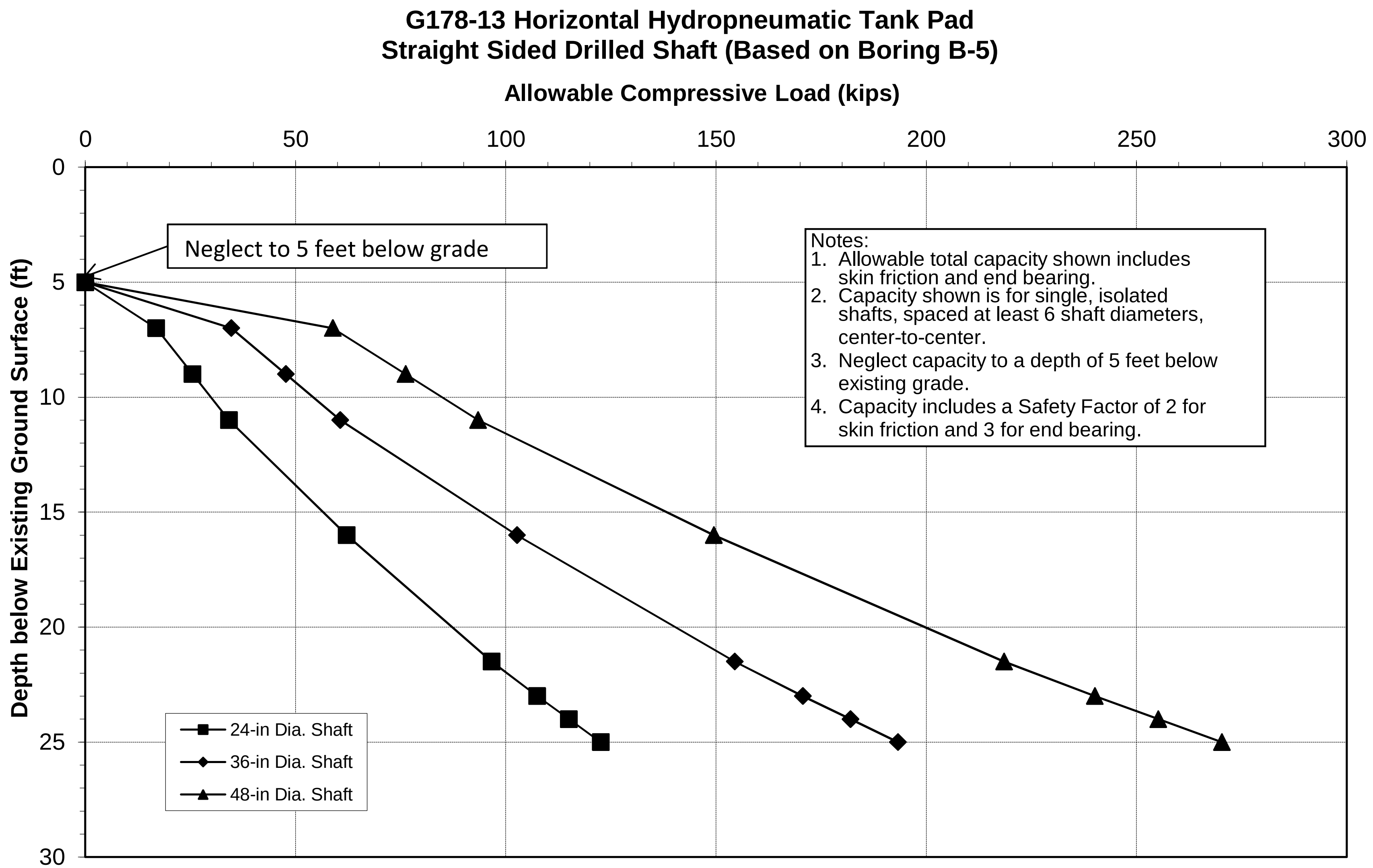

G178-13 Horizontal Hydropneumatic Tank Pad
Straight Sided Drilled Shaft (Based on Boring B-5)
Allowable Accumulative Unit Skin Friction Capacity (kips/ft diameter)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Depth below Existing Ground Surface (ft)
0
5
10
15
20
25
30
Neglect to 5 feet below grade
Notes:
1. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center (η = 1.0).
2. Neglect capacity to a depth of 5 feet below existing grade.
3. Capacity includes a Safety Factor of 2 for skin friction.
Skin Friction

Plate C-2a


G178-13 Horizontal Hydropneumatic Tank Pad
Straight Sided Drilled Shaft (Based on Boring B-5)
Allowable Unit End Bearing (ksf)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Depth below Existing Ground Surface (ft)
0
5
10
15
20
25
30
Neglect to 5 feet below grade
Notes:
1. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center (η = 1.0).
2. Neglect capacity to a depth of 5 feet below existing grade.
3. Capacity includes a Safety Factor of 3 for end bearing.
End Bearing
Plate C-2b



G178-13 Horizontal Hydropneumatic Tank Pad
Straight Sided Drilled Shaft (Based on Boring B-5)
Allowable Compressive Load (kips)
0
50
100
150
200
250
300
Depth below Existing Ground Surface (ft)
0
5
10
15
20
25
30
Neglect to 5 feet below grade
Notes:
1. Allowable total capacity shown includes skin friction and end bearing.
2. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center.
3. Neglect capacity to a depth of 5 feet below existing grade.
4. Capacity includes a Safety Factor of 2 for skin friction and 3 for end bearing.
24-in Dia. Shaft
36-in Dia. Shaft
48-in Dia. Shaft

Plate C-2c

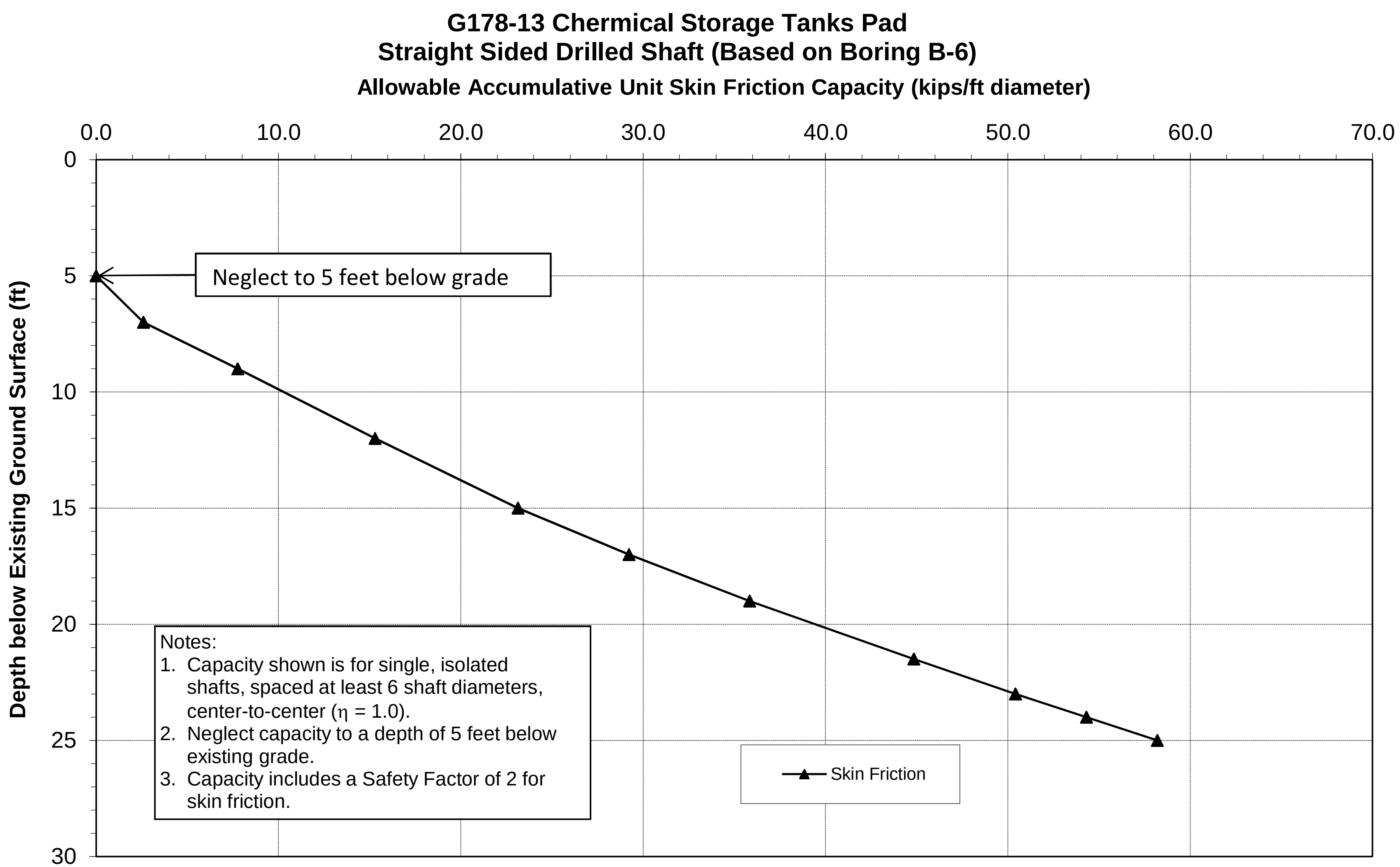

G178-13 Chermical Storage Tanks Pad
Straight Sided Drilled Shaft (Based on Boring B-6)
Allowable Accumulative Unit Skin Friction Capacity (kips/ft diameter)
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
0
5
10
15
20
25
30
Depth below Existing Ground Surface (ft)
Neglect to 5 feet below grade
Notes:
1. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center (η = 1.0).
2. Neglect capacity to a depth of 5 feet below existing grade.
3. Capacity includes a Safety Factor of 2 for skin friction.
Skin Friction

Plate C-3a

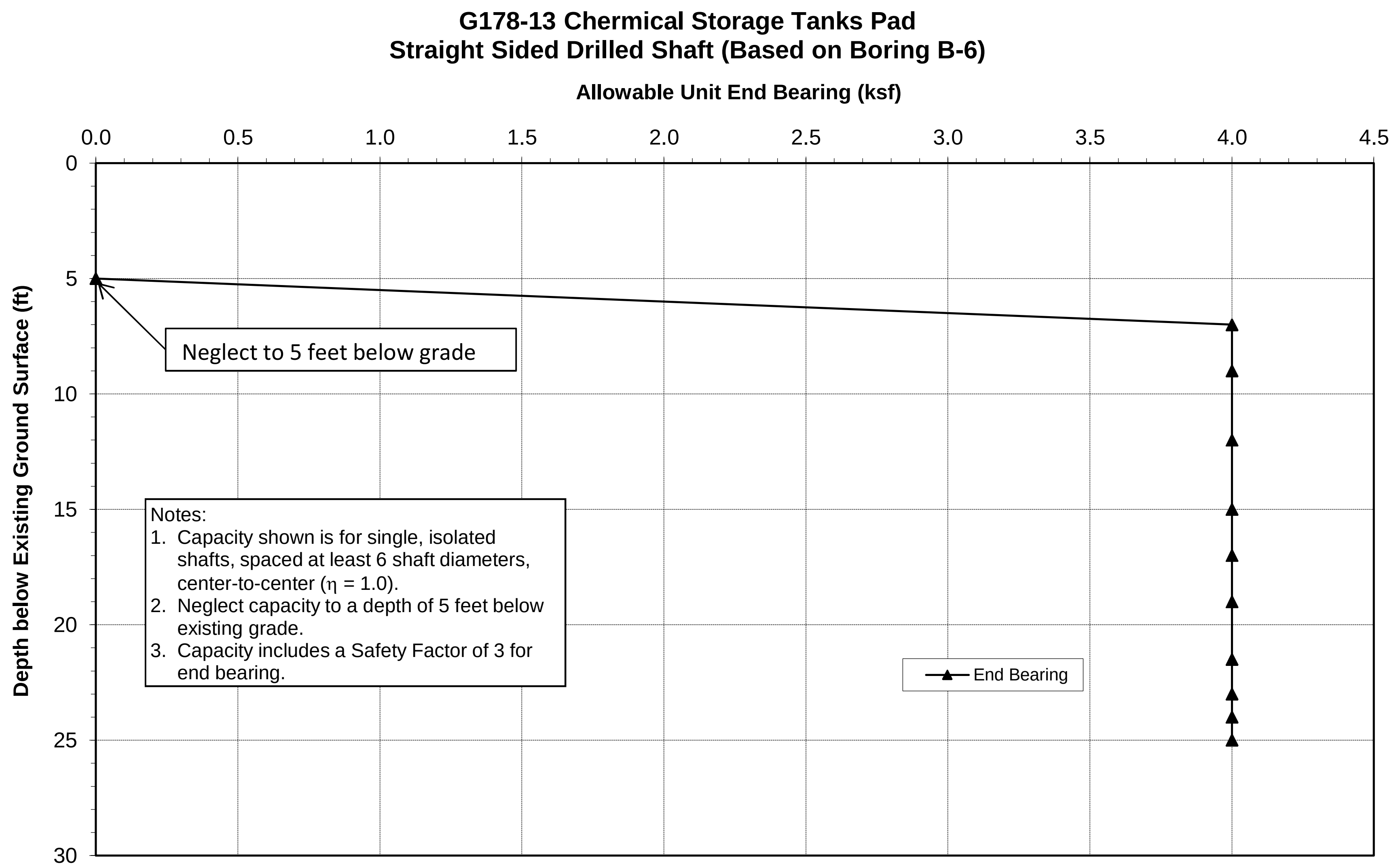

G178-13 Chermical Storage Tanks Pad
Straight Sided Drilled Shaft (Based on Boring B-6)
Allowable Unit End Bearing (ksf)
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
Depth below Existing Ground Surface (ft)
0
5
10
15
20
25
30
Neglect to 5 feet below grade
Notes:
1. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center (η = 1.0).
2. Neglect capacity to a depth of 5 feet below existing grade.
3. Capacity includes a Safety Factor of 3 for end bearing.
End Bearing

Plate C-3b

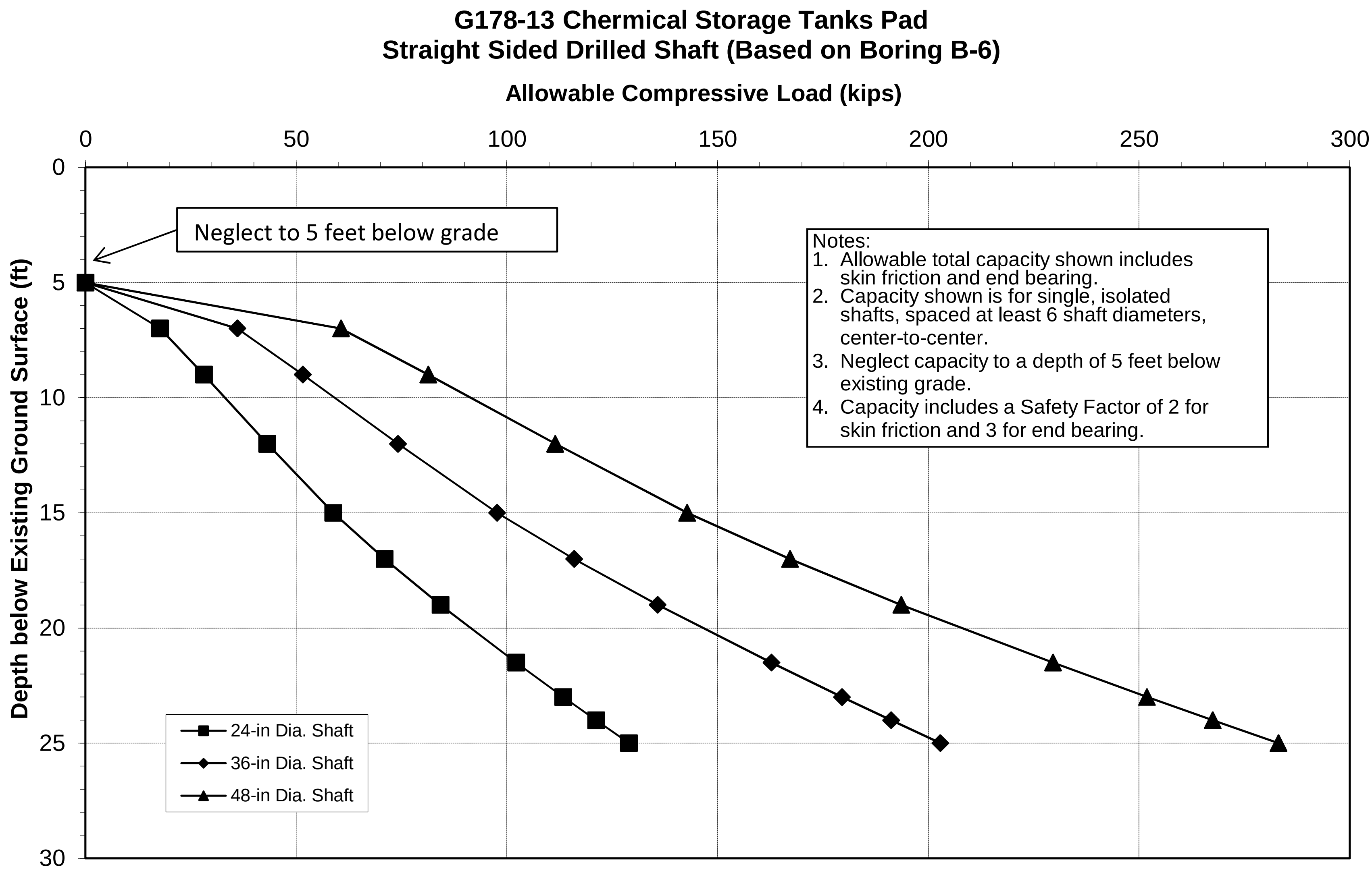

G178-13 Chermical Storage Tanks Pad
Straight Sided Drilled Shaft (Based on Boring B-6)
Allowable Compressive Load (kips)
0
50
100
150
200
250
300
Depth below Existing Ground Surface (ft)
0
5
10
15
20
25
30
Neglect to 5 feet below grade
Notes:
1. Allowable total capacity shown includes skin friction and end bearing.
2. Capacity shown is for single, isolated shafts, spaced at least 6 shaft diameters, center-to-center.
3. Neglect capacity to a depth of 5 feet below existing grade.
4. Capacity includes a Safety Factor of 2 for skin friction and 3 for end bearing.
24-in Dia. Shaft
36-in Dia. Shaft
48-in Dia. Shaft

Plate C-3c